Chapter 7   Substructure Design   Contents

7.1 General Substructure Considerations

Note that in the following guidelines where reference is made to AASHTO, the item can be found in the current AASHTO LRFD Bridge Design Specifications, with Interims.

7.1.1 Foundation Design Process

A flowchart is provided in Figure 7.1.1-1 which illustrates the overall design process needed to accomplish an LRFD foundation design. Note this process is also outlined in the Geotechnical Design Manual (GDM) in Section 8.2. The Bridge and Structures Office (BO) and the Geotechnical Branch (GB) have been abbreviated. The steps in the flowchart are defined as follows:

A. Conceptual Bridge Foundation Design

This design step results in an informal communication produced by the Geotechnical Branch at the request of the Bridge and Structures Office which provides a brief description of the following.

- Anticipated soil site conditions
- Maximum embankment slopes
- Foundation types and geotechnical hazards such as liquefaction

In general, these recommendations rely on existing site data. Site borings may not be available and test holes are drilled later. The geotechnical recommendations provide enough information to select a type of foundation for an initial Bridge Preliminary Plan.

B. Develop Site Data and Preliminary Bridge Plan

In the second phase, the Bridge and Structures Office obtains site data from the region (see WSDOT Bridge Design Manual Section 2.2) and develops the Preliminary Bridge Plan. The preliminary pier locations determine soil boring locations at this time. The Geotechnical Branch will also require the following information to continue the preliminary geotechnical design.

- Structure type and magnitude of settlement the structure can tolerate (both total and differential).
- At abutments – Approximate maximum top of foundation elevation.
- At interior piers – The number of columns; whether a single foundation element supports each column or one foundation element supports multiple columns.
- At stream crossings – Pier scour depth, if known. Typically, the Geotechnical Branch will pursue this issue with the HQ Hydraulics Section.
- Any known structural constraints that affect the foundations' type, size, or location.
- Any known constraints that affect the soil resistance (utilities, construction staging, excavation, shoring and falsework).

C. Preliminary Foundation Design

The third phase is a request by the Bridge and Structures Office for a preliminary foundation memorandum. The Geotechnical Branch memo will provide preliminary soil data required for structural analysis and modeling. This includes any subsurface conditions and the preliminary subsurface profile.

The concurrent geotechnical work at this stage includes:
- Completion of detailed boring logs and laboratory test data
- Development of foundation type, soil capacity, and foundation depth
- Development of static/seismic soil properties and ground acceleration
- Recommendations for constructability issues

D. Structural Analysis and Modeling

In the fourth phase, the Bridge and Structures Office performs a structural analysis of the superstructure and substructure using a bridge model and preliminary soil parameters. Through this modeling, the designer determines loads and sizes for the foundation based on the controlling LRFD limit states. Structural and geotechnical design continues to investigate constructability and construction staging issues during this phase.

In order to produce a Final Geotechnical Report, the Bridge and Structures Office provides the following structural feedback to the Geotechnical Engineer.
- Foundation loads for service limit state and strength limit state.
- Foundation size/diameter and depth required to meet structural design.
- Foundation details that could affect the geotechnical design of the foundations.
- Size and configuration for deep foundation groups.

E. Final Foundation Design

The last phase completes the geotechnical report and allows the final structural design to begin. The preliminary geotechnical assumptions are checked and recommendations are modified, if necessary. The final report is complete to a PS&E format since the Project Contract will contain referenced information in the Geotechnical Report, such as:
- All geotechnical data obtained at the site (boring logs, subsurface profiles, and laboratory test data)
- All final foundation recommendations
- Final constructability and staging recommendations

The designer reviews the final report for new information and confirms the preliminary assumptions. With the foundation design complete, the final bridge structural design and detailing process continues to prepare the Bridge Plans. Following final structural design, the structural designer should follow up with the geotechnical designer to ensure that the design is within the limits of the Geotechnical Report.
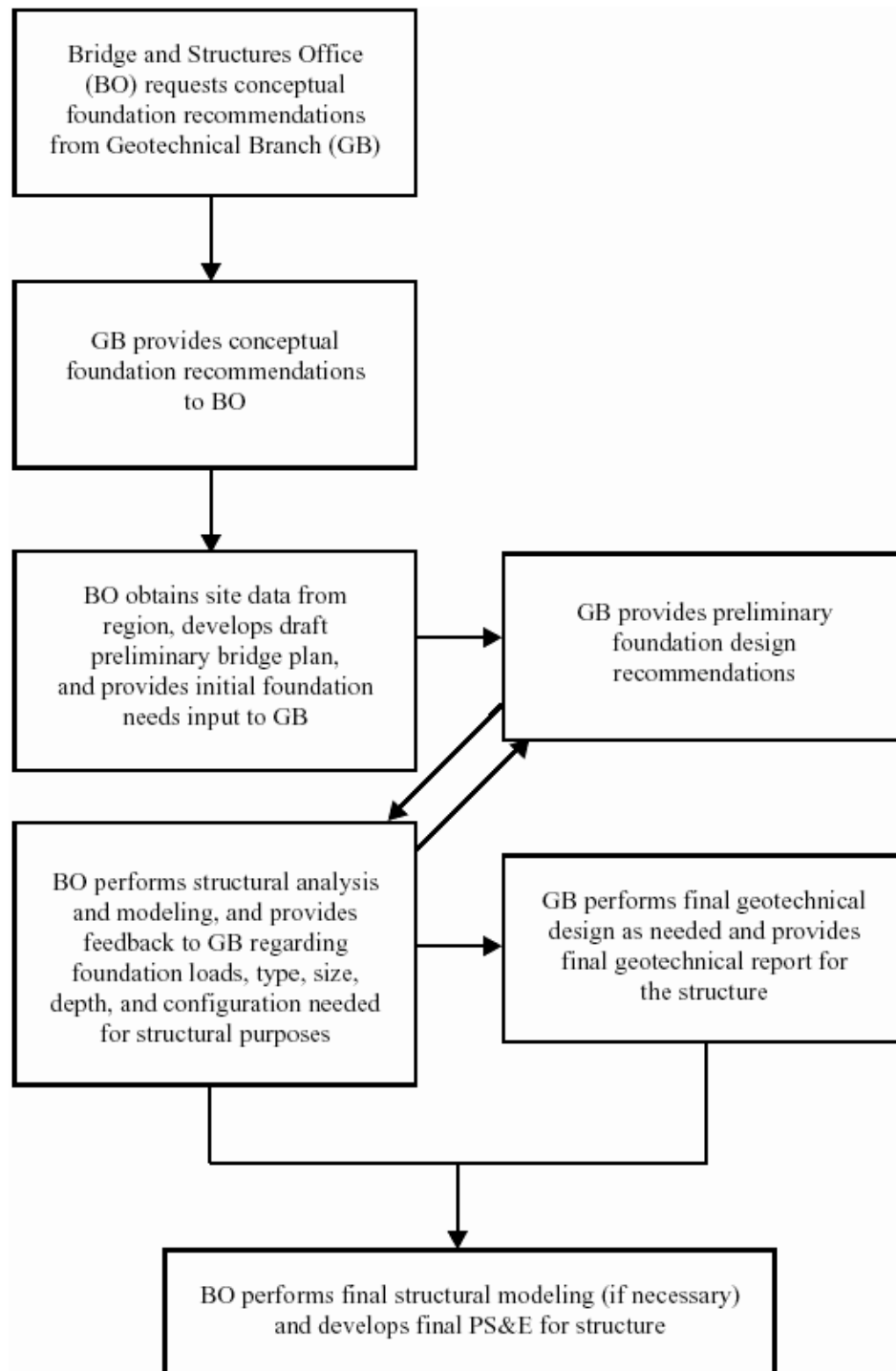
Overall Design Process for LRFD Foundation Design
Figure 7.1.1-1

7.1.2 Foundation Design Limit States

The controlling limit states for WSDOT projects for Substructure Design are described as follows:

Strength I   Relating to the normal vehicular use

Strength III   Relating to the bridge exposed to wind

Strength IV   Relating to temperature fluctuations, creep, and shrinkage

Strength V   Relating to the normal vehicular use and wind

Extreme-Event I   Relating to earthquake

Service I   Relating to normal operational use and wind

7.1.3 Seismic Design

The seismic design of all substructures shall be in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design of May 2007, or later edition, except as noted otherwise.

7.1.4 Substructure and Foundation Loads

Figure 7.1.4-1 below provides a common basis of understanding for load location and orientations for substructure design. This figure also shows elevations required for abutment and substructure design. Note that for shaft and some pile foundation designs, the shaft or pile may form the column as well as the foundation element.

Spread footings usually have a design orientation normal to the footing. Since bridge loads are longitudinal and transverse, skewed superstructure loads are converted (using vector components) to normal and parallel footing loads. Deep foundation analysis usually has a normal/parallel orientation to the pier in order to simplify group effects.
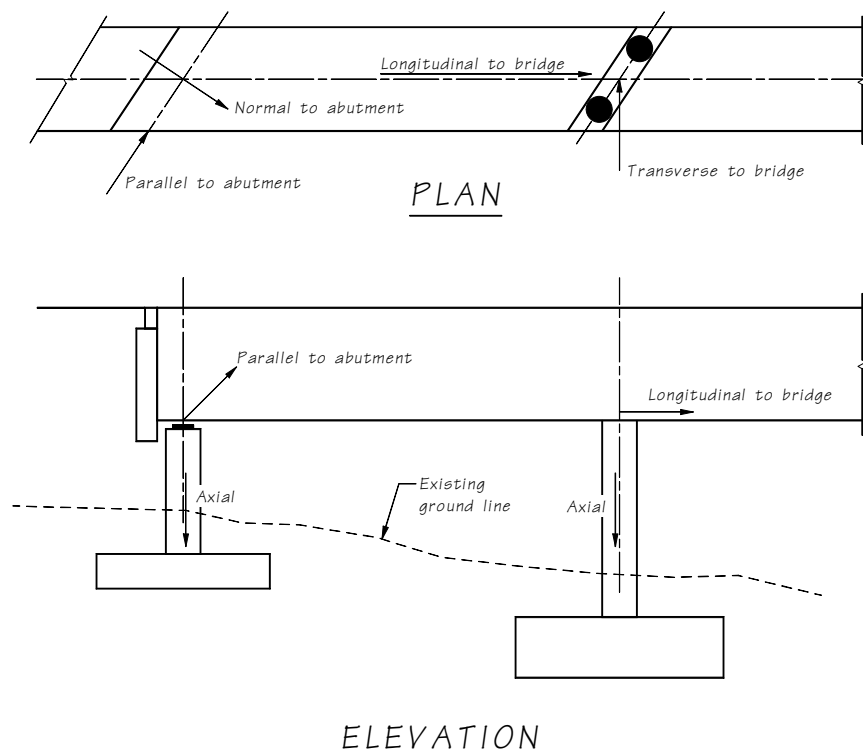
Substructure Directional Forces
Figure 7.1.4-1

Substructure elements are to carry all of the loads specified in AASHTO. Selecting the controlling load conditions requires good judgment to minimize design time.

Bridge design will consider construction loads to ensure structural stability and prevent members from overstress. For example, temporary construction loads caused by placing all of the precast girders on one side of a crossbeam can overload a single column pier. The plans shall show a construction sequence and/or notes to avoid unacceptable loadings.

On curved bridges, the substructure design shall consider the eccentricity resulting from the difference in girder lengths. When superstructure design uses a curved girder theory, such as the V-Load Method, the reactions from such analysis must be included in the loads applied to the substructure.

A. Dead Loads - DC

    Substructure design shall account for all anticipated dead load conditions. Sidesway effect shall be included where it tends to increase stresses.

B. Live Loads - LL

    The dynamic allowance (IM) shall be applied in accordance with AASHTO 3.6.2 and is not included in the design of buried elements of the substructure. Portions of the abutments in contact with the soil are considered buried elements.

D. Earthquake Loads - EQ

    Earthquake loads shall be developed in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

7.1.5   Concrete Class for Substructure

The concrete class for all substructure elements shall normally be Class 4000. This includes footings, pedestals, massive piers, columns, crossbeams, traffic barriers, and retaining walls, wingwalls, and curtain walls connected to the bridge substructure or superstructure. Foundation seals shall be Class 4000W.

7.1.6   Foundation Seals

A concrete seal within the confines of a cofferdam permits construction of a pier footing and column in the dry. This type of underwater construction is practical to a water depth of approximately 50 feet.

Seal concrete is placed underwater with the use of a tremie. A tremie is a long pipe that extends to the bottom of the excavation and permits a head to be maintained on the concrete during placement. After the concrete has been placed and has obtained sufficient strength, the water within the cofferdam is removed. In Figure 7.1.6-1, some of the factors that must be considered in designing a seal are illustrated.

COFFERDAM

NORMAL HIGH WATER ELEVATION (SEAL VENT ELEVATION)

HEAD OF WATER

1'-0" *

PED.

FOOTING

SEAL

STREAM BED ELEVATION

MINIMUM COVER, USUALLY 5' (VERIFY WITH HYDRAULIC SECTION). MUST SATISFY SECTION 9.5.

SCOUR DEPTH (RECOMMENDATION FROM HYDRAULIC SECTION)

* USUALLY 1'-0" FOR DESIGN (USE 1'-0" GREATER THAN DESIGN SEAL DIMENSIONS FOR QUANTITY CALCULATIONS).
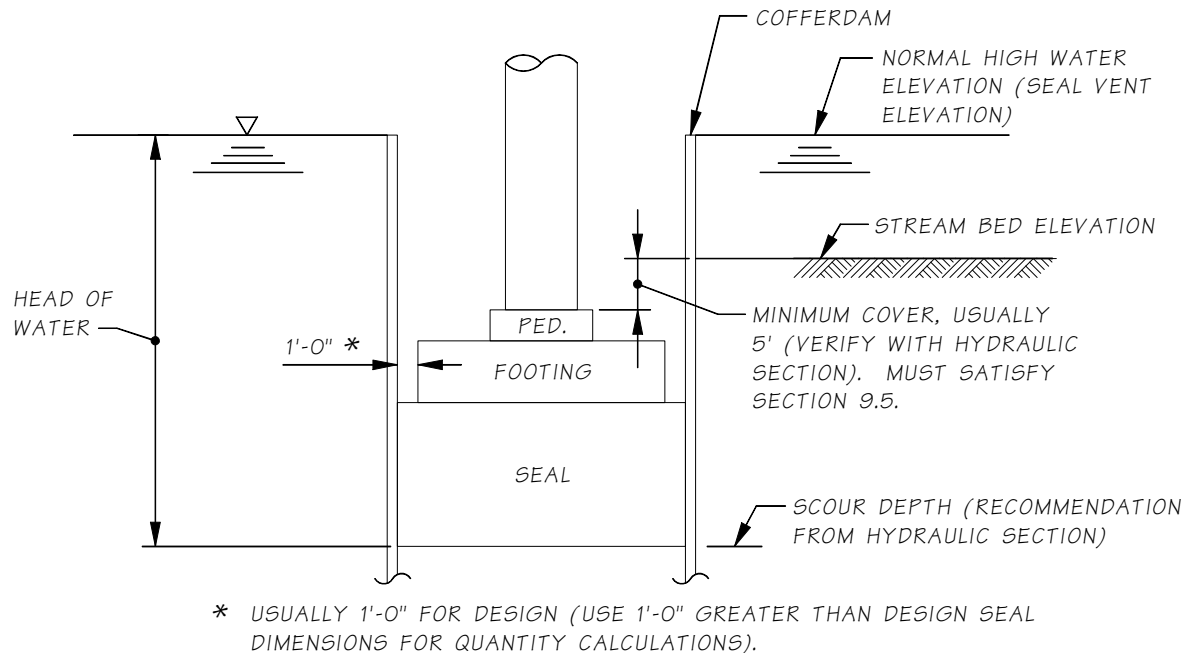
Figure 7.1.6-1

A. General Seal Criteria

The Normal High Water Elevation is defined as the highest water surface elevation that may normally be expected to occur during a given time period. This elevation, on the Hydraulics Data Sheet, is obtained from discussions with local residents or by observance of high water marks at the site. The normal high water is not related to any flood condition.

1. Seal Vent Elevation

The HQ Hydraulics Section recommends a seal vent elevation in accordance with the following criteria.

a. Construction time period not known.

If the time period of the footing construction is not known, the vent elevation reflects the normal high water elevation that might occur at any time during the year.

b. Construction time period known.

If the time period of the footing construction can be anticipated, the vent elevation reflects the normal high water elevation that might occur during this time period. (If the anticipated time period of construction is later changed, the HQ Hydraulics Section shall be notified and appropriate changes made in the design.)

2. Scour Depth

The HQ Hydraulics Section determines the depth of the anticipated scour. The bottom of footing, or bottom of seal if used, shall be no higher than the scour depth elevation. After preliminary footing and seal thicknesses have been determined, the Bridge Designer shall review the anticipated scour elevation with the Hydraulics Section to ensure that excessive depths are not used.

3. Foundation Elevation Recommended in Geotechnical Report

    Based on the results obtained from test borings at the site, the Geotechnical Engineer determines a foundation elevation, bearing capacity and settlement criteria. If other factors control, such as scour or footing cover, the final footing elevation should be adjusted as required.

4. Unusual Conditions

    Unusual site conditions such as rock formations or deep foundations require special considerations in order to obtain the most optimum design. The proposed foundation design/construction should be discussed with both the Geotechnical Branch and the HQ Hydraulics Section prior to final plan preparation.

B. Spread Footing Seals

The Geotechnical Branch will generally recommend whether a foundation seal may or may not be required for construction. Bearing loads are the column moments applied at the base of the footing and vertical load applied at the bottom of the seal. The seal is sized for the soil bearing, capacity, and Overturning Stability need only be checked at the base of the pier footing.

1. When a Seal is Required During Construction

    If the footing can be raised without violating cover requirements, the bottom of the seal elevation shall be the lower of the scour elevation or the foundation elevation as recommended by the Geotechnical Engineer. The bottom of the seal may be lower than the scour elevation or foundation elevation due to cover requirements. Spread footing final design shall include the dead load weight of the seal.

2. When a Seal May Not Be Required for Construction

    Both methods of construction are detailed in the Plans when it is not clear if a seal is required for construction. The Plans must detail a footing with a seal and an alternate without a seal. The Plan quantities are based on the footing designed with a seal. If the alternate footing elevation is different from the footing with seal, it is also necessary to note on the plans the required changes in rebar such as length of column bars, increased number of ties, etc. Note that this requires the use of either General Special Provision (GSP) 02306B1.GB6 or 02306B2.GB6.

C. Pile Footing Seals

The top of footing, or pedestal, is set by the footing cover requirements. The bottom of seal elevation is based on the stream scour elevation determined by the HQ Hydraulics Section. A preliminary analysis is made using the estimated footing and seal weight, and the column moments and vertical load at the base of the footing to determine the number of piles and spacing. The seal size shall be 1 foot 0 inches larger than the footing all around. If the seal is omitted during construction, the bottom of footing shall be set at the scour elevation and an alternate design is made.

In general seal design requires determining a thickness such that the seal weight plus any additional resistance provided by the bond stress between the seal concrete and any piling is greater than the buoyant force (determined by the head of water above the seal). If the bond stress between the seal concrete and the piling is used to determine the seal thickness, the uplift capacity of the piles must be checked against the loads applied to them as a result of the bond stress. The bond between seal concrete and piles is typically assumed to be 10 psi by other DOT's. As such it is also allowed here. The minimum seal thickness is 1'-6".

7.2   Foundation Modeling

7.2.1   General

Bridge modeling for seismic events shall be in accordance with requirements of the AASHTO Guide Specifications for LRFD Seismic Bridge Design Section 5, "Analytical Models and Procedures."

The following sections were developed for a force-based seismic design as required in previous versions of the AASHTO LRFD Bridge Design Specifications. Modifications have been made to the following sections to incorporate the provisions of the new AASHTO Guide Specifications for LRFD Seismic Bridge Design. As such, it is anticipated that this section will be revised as more experience is gained through the application of the Guide Specifications.

7.2.2   Substructure Linear Dynamic Analysis Procedure

The following is a general description of the iterative process used in a linear dynamic analysis. Note, a linear dynamic analysis is needed to determine the displacement demand, Δ_D.

1. Build a Finite Element Model (FEM) in order to determine initial forces to substructure elements (EQ+DL). Assume that foundation springs are located at the bottom of the column.

    A good initial support assumption for deep foundations (shafts or piles) would be to add 10 feet to the column length in stiff soils and 15 feet to the column in soft soils. An alternate method is to use 85% of the fixed support reactions for the initial forces. Use fully fixed forces for foundations in rock.

    Use multi-mode response spectrum analysis to generate initial Seismic Shear, Moment, & Axial Loads.

2. Using the initial forces, determine a preliminary footing size, shaft size/length, or pile group arrangement. Note, the load combinations specified in Article 4.4 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design SHOULD NOT be used in this iterative analysis.

3. For spread footing foundations, the FEM will include foundation springs calculated based on the footing size as calculated in BDM Section 7.2.7. No iteration is required unless the footing size changes.

4. For deep foundation analysis, the FEM and the soil response program must agree or converge on soil/structure lateral response. In other words, the moment, shear, deflection, and rotation of the two programs should be within 10%. More iteration will provide convergence much less than 1%. The iteration process to converge is as follows:

    a. Apply the initial FEM loads (moment and shear) to a soil response program such as DFSAP. DFSAP is a program that models Short, Intermediate or Long shafts or piles using the Strain Wedge Theory.

    b. Calculate foundation spring values for the FEM. Note, the load combinations specified in Article 4.4 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design should not be used to determine foundation springs.

    c. Re-run the seismic analysis using the foundation springs calculated from the soil response program. The structural response will change. Check to insure the FEM results (M, V, Δ, θ, and spring values) in the transverse and longitudinal direction are within 10% of the previous run. This check verifies the linear spring, or soil response (calculated by the FEM) is close to the predicted nonlinear soil behavior (calculated by the soil response program). If the results of the FEM and the soil response program differ by more than 10%, recalculate springs and repeat steps (a) thru (c) until the two programs converge to within 10%.

Special note for single column/single shaft configuration: The seismic design philosophy requires a plastic hinge in the substructure elements above ground (preferably in the columns). Designers should note the magnitude of shear and moment at the top of the shaft, if the column "zero" moment is close to a shaft head foundation spring, the FEM and soil response program will not converge and plastic hinging might be below grade.

Throughout the iteration process it is important to note that any set of springs developed are only applicable to the loading that was used to develop them (due to the inelastic behavior of the soil in the foundation program). This can be a problem when the forces used to develop the springs are from a seismic analysis that combines modal forces using a method such as the Complete Quadratic Combination (CQC) or other method. The forces that result from this combination are typically dominated by a single mode (in each direction as shown by mass participation). This results in the development of springs and forces that are relatively accurate for that structure. If the force combination (CQC or otherwise) is not dominated by one mode shape (in the same direction), the springs and forces that are developed during the above iteration process may not be accurate.

7.2.3   Bridge Model Section Properties

In general, gross section properties may be assumed for all FEM members, except concrete columns.

A.   Cracked Properties for Columns

Effective section properties shall be in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design Section 5.6.

B.   Shaft Properties

The moment of inertia for shafts shall be based on the gross section (I_g) or non-cracked. The shaft concrete strength and construction methods lead to significant variation in shaft stiffness described as follows.

For a stiff substructure response:

1.   Use 1.5 f'c to calculate the modulus of elasticity. Since aged concrete will generally reach a compressive strength of at least 6 ksi when using a design strength of 4 ksi, the factor of 1.5 is a reasonable estimate for an increase in stiffness.

2.   Increase shaft I_g by increasing the shaft diameter by the amount allowed by the current ADSC/WSDOT Shaft Special Provision. The ADSC/WSDOT shaft special provision allows contractors to increase the shaft diameter to accommodate metric casings used in the oscillator and rotator drilling methods. See subsection 2.01.D of the current ADSC/WSDOT Shaft Special Provision.

3.   In cases where stepped shaft construction is allowed, the specified increase in diameter should be included in the bracketing of the response. See subsection 3.03.C of the current ADSC/WSDOT Shaft Special Provision for allowable increase in shaft diameter for stepped shafts with telescoping casing.

4.   When permanent casing is specified, increase shaft I_g using the transformed area of a ¾" thick casing. Since the contractor will determine the thickness of the casing, ¾" is a conservative estimate for design.

For a soft substructure response:

1. Use 0.85 f'c to calculate the modulus of elasticity. Since the quality of shaft concrete can be suspect when placed in water, the factor of 0.85 is an estimate for a decrease in stiffness.

2. Use shaft I_g.

3. When permanent casing is specified, increase shaft I_g using the transformed area of a ⅜" thick casing. Since the contractor will determine the thickness of the casing, ⅜" is a minimum estimated thickness for design.

C. Cast-in-Place Pile Properties

For a stiff substructure response:

1. Use 1.5 f'c to calculate the modulus of elasticity. Since aged concrete will generally reach a compressive strength of at least 6 ksi when using a design strength of 4 ksi, the factor of 1.5 is a reasonable estimate for an increase in stiffness.

2. Use the pile I_g plus the transformed casing moment of inertia. Note: If DFSAP is used for analysis, the reinforcing and shell properties are input and the moment of inertia is computed internally.

$$I_{pile} = I_g + (n)(I_{shell}) + (n)(I_{reinf})$$
where: $n = E_c/E_s$

Use a steel casing thickness of ¼" for piles less than 14 inches in diameter, ⅜" for piles 14 to 18 inches in diameter, and ½" for larger piles. Note: These casing thicknesses are to be used for analysis only, the contractor is responsible for selecting the casing thickness required to drive the piles.

For a soft substructure response:

1. Use 1.0 f'c to calculate the modulus of elasticity.

2. Use pile I_g, neglecting casing properties.

7.2.4   Bridge Model Verification

As with any FEM, the designer should review the foundation behavior to ensure the foundation springs correctly imitate the known boundary conditions and soil properties. Watch out for mismatch of units.

All finite element models must have dead load static reactions verified and boundary conditions checked for errors. The static dead loads (DL) must be compared with hand calculations or another program's results. For example, span member end moment at the supports can be released at the piers to determine simple span reactions. Then hand calculated simple span DL or PGsuper DL and LL is used to verify the model.

Crossbeam behavior must be checked to ensure the superstructure DL is correctly distributing to substructure elements. A 3D bridge line model concentrates the superstructure mass and stresses to a point in the crossbeam. Generally, interior columns will have a much higher loading than the exterior columns. To improve the model, crossbeam I_g should be increased to provide the statically correct column DL reactions. This may require increasing I_g by about 1000 times. Many times this is not visible graphically and should be verified by checking numerical output.

Seismic analysis may also be verified by hand calculations. Hand calculated fundamental mode shape reactions will be approximate; but will ensure design forces are of the same magnitude.

Designers should note that additional mass might have to be added to the bridge FEM for seismic analysis. For example, traffic barrier mass and crossbeam mass beyond the last column at piers may contribute significant weight to a two-lane or ramp structure.

7.2.5   Deep Foundation Modeling Methods

A designer must assume a foundation support condition that best represents the foundation behavior. Deep foundation elements attempt to imitate the non-linear lateral behavior of several soil layers interacting with the deep foundation. The bridge FEM then uses the stiffness of the element to predict the seismic structural response. Models using linear elements that are not based on non-linear soil-structure interaction are generally considered inaccurate for soil response/element stress and are not acceptable. There are three methods used to model deep foundations (FHWA Report No. 1P-87-6). Of these three methods the Bridge and Structures Office prefers Method II for the majority of bridges.

A. Method I – Equivalent Cantilever Column

This method assumes a point of fixity some depth below the bottom of the column to model the stiffness of the foundation element. This could be used for a preliminary model of the substructure response.

B. Method II – Equivalent Base Springs

This method models deep foundations by using a {6x6} matrix. There are two techniques used to generate the stiffness coefficients for the foundation matrix. The equivalent stiffness coefficients assessed are valid only at the given level of loading. Any changes of the shaft-head loads or conditions will require a new run for the program to determine the new values of the equivalent stiffness coefficients. These equivalent stiffness coefficients account for the nonlinear response of shaft materials and soil resistance.

Technique I – The matrix is generated, using superposition, to reproduce the non-linear behavior of the soil and foundation at the maximum loading. With Technique I, 10 terms are produced, 4 of these terms are "cross couples". Soil response programs, such as Lpile or DFSAP, analyze the non-linear soil response. The results are then used to determine the equivalent base springs. See Appendix 7-B-1 for more information

Technique II – The equivalent stiffness matrix generated using this technique uses only the diagonal elements (no cross coupling stiffnesses). The DFSAP program should be used to develop the equivalent stiffness matrix. This technique is recommended be used to construct the foundation stiffness matrix (equivalent base springs).

In Technique II the "cross couple" effects are internally accounted for as each stiffness element and displacement is a function of the given Lateral load (P) and Moment (M). Technique II uses the total response ($\Delta_{t(P,M)}$ $\theta_{t(P,M)}$) to determine displacement and equivalent soil stiffness, maintaining a nonlinear analysis. Technique I requires superposition by adding the individual responses due to the lateral load and moment to determine displacement and soil stiffness. Using superposition to combine two nonlinear responses results in errors in displacement and stiffness for the total response as seen in the Figure 7.2.5-1. As illustrated, the total response due to lateral load (P) and moment (M) does not necessarily equal the sum of the individual responses. For more details on the equivalent stiffness matrix, see the DFSAP reference manual.
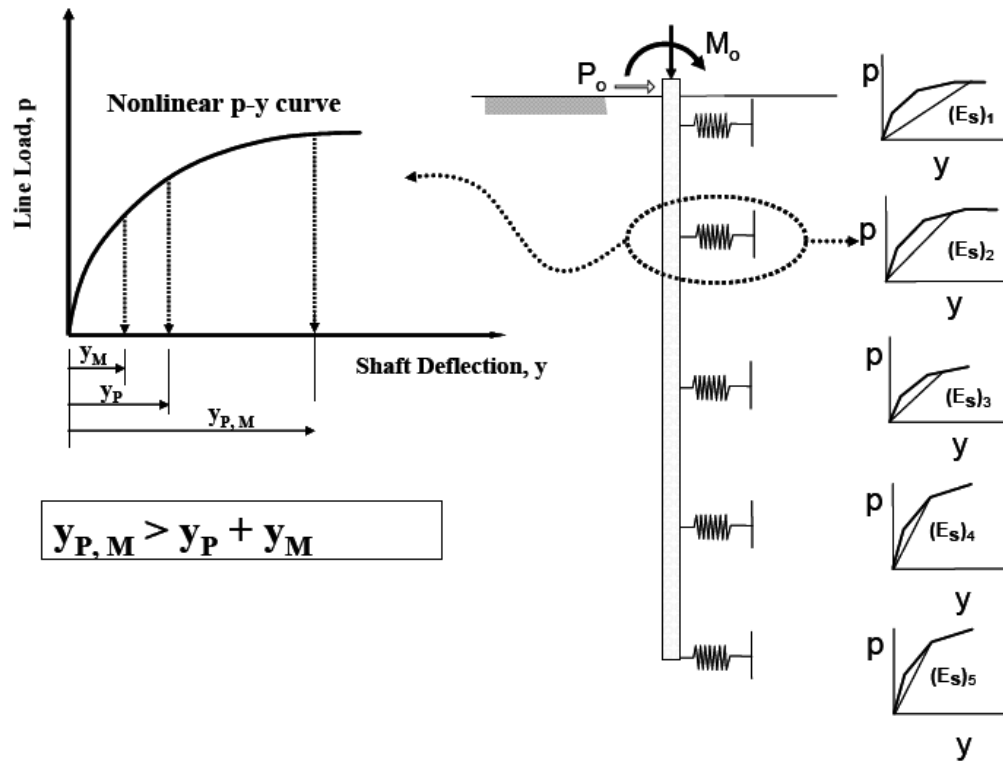
Limitations on the Technique I (Superposition Technique)
Figure 7.2.5-1

C. Method III – Non-Linear Soil Springs

This method attaches non-linear springs along the length of deep foundation members in a FEM model. See Appendix 7-B-2 for more information. This method has the advantage of solving the superstructure and substructure seismic response simultaneously. The soil springs must be nonlinear PY curves and represent the soil/structure interaction. This cannot be done during dynamic analysis with some FEM programs (including GTStrudl).

D. Spring Location (Method II)

The preferred location for a foundation spring is at the bottom of the column. This includes the column mass in the seismic analysis. For design, the column forces are provided by the FEM and the soil response program provides the foundation forces. Springs may be located at the top of the column. However, the seismic analysis will not include the mass of the columns. The advantage of this location is the soil/structure analysis includes both the column and foundation design forces.

Designers should be careful to match the geometry of the FEM and soil response program. If the location of the foundation springs (or node) in the FEM does not match the location input to the soil response program, the two programs will not converge correctly.

E. Boundary Conditions (Method II)

To calculate spring coefficients, the designer must first identify the predicted shape, or direction of loading, of the foundation member where the spring is located in the bridge model. This will determine if one or a combination of two boundary conditions apply for the transverse and longitudinal directions of a support.

A fixed head boundary condition occurs when the foundation element is in double curvature where translation without rotation is the dominant behavior. Stated in other terms, the shear causes deflection in the opposite direction of applied moment. This is a common assumption applied to both directions of a rectangular pile group in a pile supported footing.

A free head boundary condition is when the foundation element is in single curvature where translation and rotation is the dominant behavior. Stated in other terms, the shear causes deflection in the same direction as the applied moment. Most large diameter shaft designs will have a single curvature below ground line and require a free head assumption. The classic example of single curvature is a single column on a single shaft. In the transverse direction, this will act like a flagpole in the wind, or free head. What is not so obvious is the same shaft will also have single curvature in the longitudinal direction (below the ground line), even though the column exhibits some double curvature behavior. Likewise, in the transverse direction of multi-column piers, the columns will have double curvature (frame action). The shafts will generally have single curvature below grade and the free head boundary condition applies. The boundary condition for large shafts with springs placed at the ground line will be free head in most cases.

The key to determine the correct boundary condition is to resolve the correct sign of the moment and shear at the top of the shaft (or point of interest for the spring location). Since multi-mode results are always positive (CQC), this can be worked out by observing the seismic moment and shear diagrams for the structure. If the sign convention is still unclear, apply a unit load in a separate static FEM run to establish sign convention at the point of interest.

The correct boundary condition is critical to the seismic response analysis. For any type of soil and a given foundation loading, a fixed boundary condition will generally provide soil springs four to five times stiffer than a free head boundary condition.

F. Spring Calculation (Method II)

The first step to calculate a foundation spring is to determine the shear and moment in the structural member where the spring is to be applied in the FEM. Foundation spring coefficients should be based on the maximum shear and moment from the applied longitudinal OR transverse seismic loading. The combined load case (1.0L and 0.3T) should be assumed for the design of structural members, and NOT applied to determine foundation response. For the simple case of a bridge with no skew, the longitudinal shear and moment are the result of the seismic longitudinal load, and the transverse components are ignored. This is somewhat unclear for highly skewed piers or curved structures with rotated springs, but the principle remains the same.

G. Matrix Coordinate Systems (Method II)

The Global coordinate systems used to demonstrate matrix theory are usually similar to the system defined for substructure loads in BDM Section 7.1.3, and is shown in Figure 7.2.5-2. This is also the default Global coordinate system of GTStrudl. This coordinate system applies to this BDM Section to establish the sign convention for matrix terms. Note vertical axial load is labeled as P, and horizontal shear load is labeled as V.
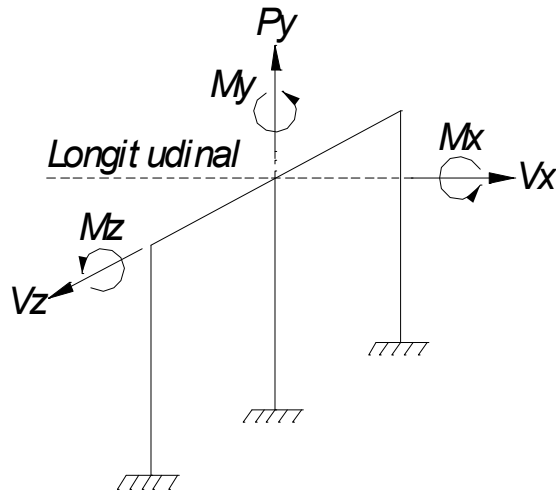
Global Coordinate System
Figure 7.2.5-2

H. Matrix Coefficient Definitions (Method II)

The stiffness matrix containing the spring values and using the standard coordinate system is shown in Figure 7.2.5-3. (Note that cross-couple terms generated using Technique I are omitted). For a description of the matrix generated using Technique I see Appendix 7-B-1. The coefficients in the stiffness matrix are generally referred to using several different terms. Coefficients, spring or spring value are equivalent terms. Lateral springs are springs that resist lateral forces. Vertical springs resist vertical forces.

$$
\begin{Bmatrix}
 & Vx & Py & Vz & Mx & My & Mz \\
Vx & K11 & 0 & 0 & 0 & 0 & 0 \\
Py & 0 & K22 & 0 & 0 & 0 & 0 \\
Vz & 0 & 0 & K33 & 0 & 0 & 0 \\
Mx & 0 & 0 & 0 & K44 & 0 & 0 \\
My & 0 & 0 & 0 & 0 & K55 & 0 \\
Mz & 0 & 0 & 0 & 0 & 0 & K66
\end{Bmatrix}
\times
\begin{Bmatrix}
\text{Disp.} \\ \Delta x \\ \Delta y \\ \Delta z \\ \theta x \\ \theta y \\ \theta z
\end{Bmatrix}
=
\begin{Bmatrix}
\text{Force} \\ Vx \\ Py \\ Vz \\ Mx \\ My \\ Mz
\end{Bmatrix}
$$

Standard Global Matrix
Figure 7.2.5-3

Where the linear spring constants or K values are defined as follows, using the Global Coordinates:

K11 = Longitudinal Lateral Stiffness (kip/in)
K22 = Vertical or Axial Stiffness (kip/in)
K33 = Transverse Lateral Stiffness (kip/in)
K44 = Transverse Bending or Moment Stiffness (kip-in/rad)
K55 = Torsional Stiffness (kip-in/rad)
K66 = Longitudinal Bending or Moment Stiffness (kip-in/rad)

The linear lateral spring constants along the diagonal represent a point on a non-linear soil/ structure response curve. The springs are only accurate for the applied loading and less accurate for other loadings. This is considered acceptable for Strength and Extreme Event design. For calculation of spring constants for Technique I see Appendix 7-B-1. For calculation of spring constants for Technique II see the DFSAP reference manual.

I.   Group Effects

When a foundation analysis uses Lpile or an analysis using PY relationships, group effects will require the geotechnical properties to be reduced before the spring values calculated. The Geotechnical Report will provide transverse and longitudinal multipliers that are applied to the PY curves. This will reduce the pile resistance in a linear fashion. The reduction factors for lateral resistance due to the interaction of deep foundation members is provided in the WSDOT Geotechnical Design Manual, Section 8.12.2.5.

Group effect multipliers are not valid when the DFSAP program is used. Group effects are calculated internally using Strain Wedge Theory.

J.   Shaft Caps and Pile Footings

Where pile supported footings or shaft caps are entirely below grade, their passive resistance should be utilized. In areas prone to scour or lateral spreading, their passive resistance should be neglected. DFSAP has the capability to account for passive resistance of footings and caps below ground.

7.2.6   Lateral Analysis of Piles and Shafts

Lateral analysis of piles and shafts SHOULD NOT be based on the load combinations described in Article 4.4 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

In general, lateral analysis of piles and shafts involves determination of a shaft or pile tip location sufficient to resist lateral loads. In many cases, the shaft or pile tip depth required to resist lateral loads may be deeper than that required for bearing or uplift. Determination of shaft or pile tip location requires engineering judgment. Suggested criteria are:

A.   For seismically controlled designs, the bigger (stiffer) the shaft the more movement can be tolerated at the shaft tip. A seismic analysis will predict the maximum deflections and stresses and the engineer must determine a safe shaft depth to survive the event. Small pile fixity generally refers to the point of fixity for column design, or the first inflection point when observing deflection. The pile tip for small shafts/piles, one to two feet in diameter, should be determined at the location of approximately the second point of inflection. An acceptable movement at the tip during an Extreme Event has yet to be determined. In general, the smaller the better. Since these shafts/piles are relatively flexible in the soil, it is possible to have pile tips at the 2nd point of inflection with little or no movement (drift) and not have deep tip elevations that are costly.

B.   Medium sized shafts, three to eight feet in diameter, tipping the shafts should consider an elevation near the midpoint of the 1st and 2nd inflection points. An acceptable movement at the tip during an Extreme Event has yet to be determined. In general, past practice has been the smaller the better based on the experience of small flexible piles.

C.   Large shafts, greater than 10-foot diameter, will transfer significantly more stresses to the soil and much deeper in the soil than flexible piles. Tipping for large shafts should consider an elevation between the midpoint and near the quarter point of the 1st and 2nd inflections. An acceptable movement at the tip during an Extreme Event has yet to be determined.

The static parameters represent the soil behavior for short-term transient loads such as wind, ice, temperature, and vessel impact. For earthquake loads, the seismic and static soil properties will be the same if the soils present have a stiffness which does not degrade with time during shaking.

If liquefiable soils are present, both static and liquefied soil properties are provided in the Geotechnical Report. Often, the highest acceleration the bridge sees is in the first cycles of the earthquake, and liquefaction tends to occur toward the middle or end of the earthquake. Therefore, early in the earthquake, loads are high, soil-structure stiffness is high, and deflections are low. Later in the earthquake, the soil-structure stiffness is lower and deflections higher.

If liquefaction can occur, the bridge should be analyzed twice. The first analysis uses the static soil conditions, which yields higher moment and shear to design the shaft (and column). The second analysis uses the liquefied soils to evaluate the bridge Extreme Event deflections. The intent here is to bracket the structure response. The designer will have to determine the acceptable maximum lateral deflection.

7.2.7   Spread Footing Modeling

For a first trial footing configuration, Strength column moments or column plastic hinging moments may be applied to generate footing dimensions. Soil spring constants are developed using the footing plan area, embedment depth, Poisson's ratio ν shear modulus G. See Article 5.3.2 in the AASHTO Guide Specifications for LRFD Seismic Bridge Design for determination of G. Spring constants for shallow rectangular footings are obtained by modifying circular footing theory using the following Equation. This method for calculating footing springs is referenced in FHWA-IP-87-6, Section 7.2.4A, page 140.

$K = \alpha\beta\, K_o$

K = Rotational or Lateral spring

K_o = Stiffness coefficient for the equivalent circular footing, see Figure 7.2.7-1. These values are calculated using an equivalent circular footing radius. See Figure 7.2.7-2

α = Foundation shape correction factor, see Figure 7.2.7-3

β = Embedment factor, see Figure 7.2.7-4

Displacement Degree-of-Freedom	K_o
Vertical translation	$\dfrac{4GR}{1-\nu}$
Horizontal translation	$\dfrac{8GR}{2-\nu}$
Torsional rotation	$\dfrac{16GR^3}{3}$
Rocking rotation	$\dfrac{8GR^3}{3(1-\nu)}$

Stiffness Coefficients
Figure 7.2.7-1

Figure 7.2.7-2 describes the parameters used to calculate the equivalent radius values (R). Note, that "D" is the depth, or thickness, of the footing. "D" is not the total embedment of the footing (the distance from the ground line to the bottom of the footing).
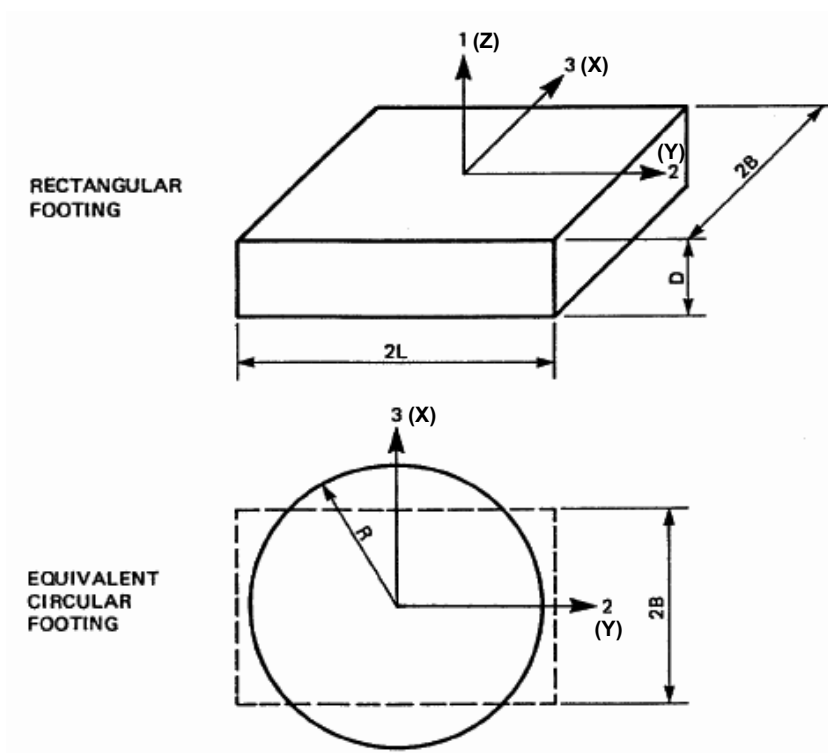
Equivalent Radius	K_o
Translational	$R_o = \sqrt{\dfrac{4BL}{\pi}}$
Z-Axis Torsion	$R_1 = \left[\dfrac{4BL(4B^2 + 4L^2)}{6\pi}\right]^{1/4}$
Y-Axis Rocking	$R_2 = \left[\dfrac{(2B)^3(2L)}{3\pi}\right]^{1/4}$
X-Axis Rocking	$R_3 = \left[\dfrac{(2B)(2L)^3}{3\pi}\right]^{1/4}$

Stiffness Coefficients
Figure 7.2.7-2
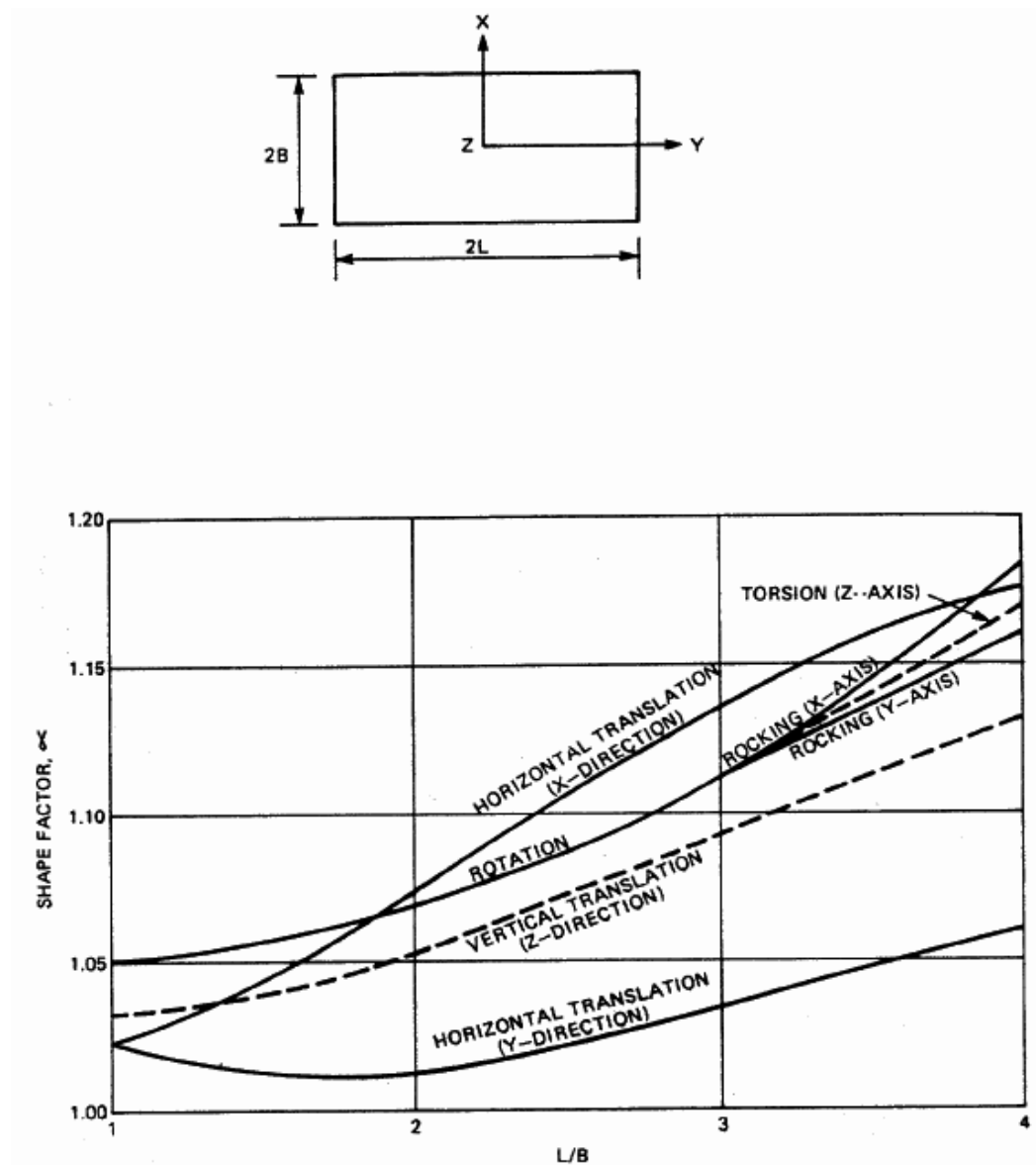
Shape Factor for Rectangular Footings
Figure 7.2.7-3

Embedment Factor
Figure 7.2.7-4

7.3 Column Design

7.3.1 Preliminary Plan Stage

The preliminary plan stage determines the initial column size, column spacing, and bridge span length based on a preliminary analysis. Columns are spaced to give maximum structural benefit except where aesthetic considerations dictate otherwise. Piers normally are spaced to meet the geometric and aesthetic requirements of the site and to give maximum economy for the total structure. Good preliminary engineering judgment results in maximum economy for the total structure.

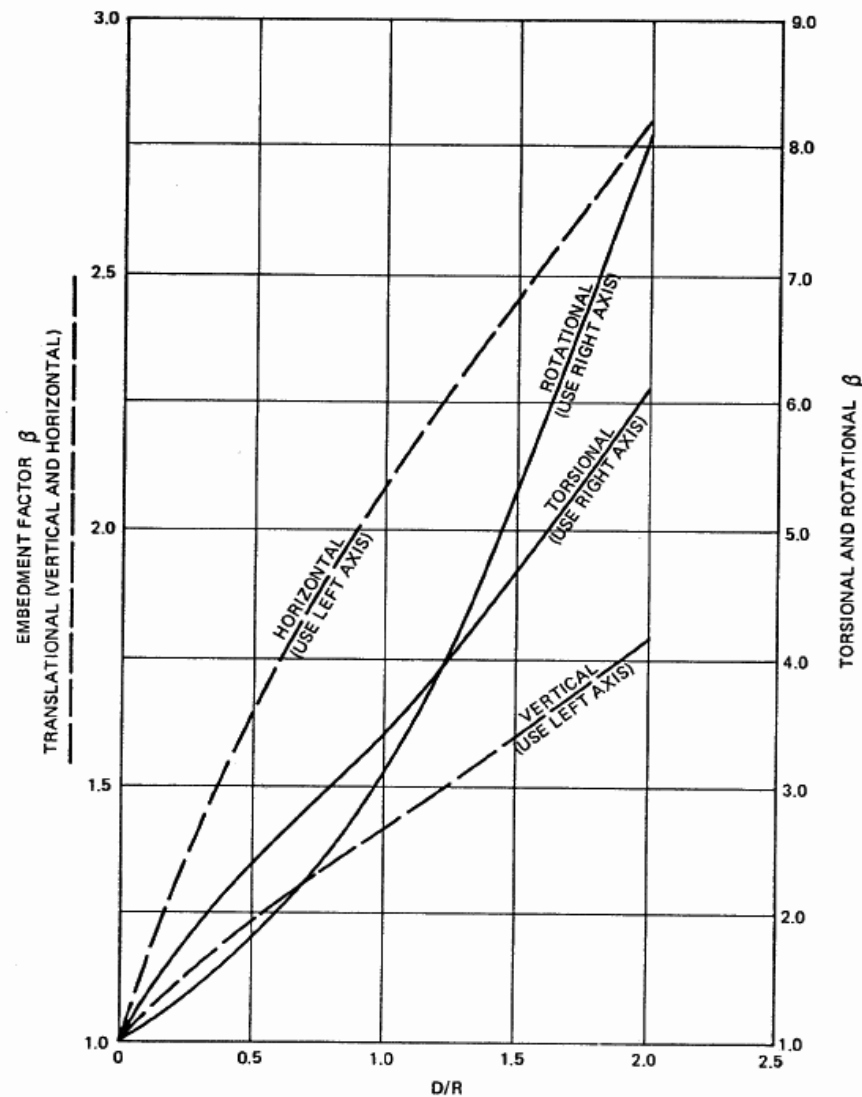The designer may make changes after the preliminary plan stage. The design unit supervisor will need to review all changes, and if the changes are more than minor dimension adjustments, the Bridge Projects Engineer and the State Bridge and Structures Architect will also need to be involved in the review.

Tall piers spaced farther apart aesthetically justify longer spans. Difficult and expensive foundation conditions will also justify longer spans. Span lengths may change in the design stage if substantial structural improvement and/or cost savings can be realized. The designer should discuss the possibilities of span lengths or skew with the supervisor as soon as possible. Changes in pier spacing at this stage can have significant negative impacts to the geotechnical investigation.

Column spacing should minimize column dead load moments. Multiple columns are better suited for handling lateral loads due to wind and/or earthquake. The designer may alter column size or spacing for structural reasons or change from a single-column pier to a multicolumn pier.

7.3.2 General Column Criteria

Columns should be designed so that construction is as simple and repetitious as possible. The diameter of circular columns should be a multiple of one foot. Rectangular sections should have lengths and widths that are multiples of 3 inches. Long rectangular columns are often tapered to reduce the amount of column reinforcement required for strength. Tapers should be linear for ease of construction.

Understanding the effects on long columns due to applied loads is fundamental in their design. Loads applied to the columns consist of reactions from loads applied to the superstructure and loads applied directly to the columns. For long columns, it may be advantageous to reduce the amount of reinforcement as the applied loads decrease along the column. In these cases, load combinations need to be generated at the locations where the reinforcement is reduced.

A. Construction Joints

Bridge Plans shall show column construction joints at the top of footing or pedestal and at the bottom of crossbeam. Optional construction joints with roughened surfaces should be provided at approximately 30-foot vertical spacing.

B. Modes of Failure

A column subject to axial load and moment can fail in several modes. A "short" column can fail due to crushing of the concrete or to failure of the tensile reinforcement. A "long" column can fail due to elastic buckling even though, in the initial stages, stresses are well within the normal allowable range. Long column failure is normally a combination of stability and strength failure that might occur in the following sequence:

1. Axial load is applied to the column.

2. Bending moments are applied to the column, causing an eccentric deflection.

3. Axial loads act eccentrically to the new column center line producing P-Δ moments which add directly to the applied moments.

4. P-Δ moments increase the deflection of the column and lead to more eccentricity and moments.

5. The P-Δ analysis must prove the column loading and deflection converges to a state where column stresses are acceptable. Otherwise, the column is not stable and failure can be catastrophic.

C. Bridge vs. Building Columns

Unlike building columns, bridge columns are required to resist lateral loads through bending and shear. As a result, these columns may be required to resist relatively large applied moments while carrying nominal axial loads. In addition, columns are often shaped for appearance. This results in complicating the analysis problem with non-prismatic sections.

7.3.3   Column Design Flow Chart – Non-Seismic Design

Figure 7.3.3-1 illustrates the basic steps in the column design process for non-seismic design.

K = EFFECTIVE LENGTH FACTOR
ℓu = UNSUPPORTED LENGTH
Kℓu = EFFECTIVE LENGTH

DETERMINE BASIC
COLUMN DATA
LOADS, ℓu, SIZE,
SUPPORTS, BRACING

CHART OF
DESIGN FUNCTIONS

ESTABLISH "K",
COMPUTE $\dfrac{K\ell u}{r}$

$\dfrac{K\ell u}{r} \leq 100$

$\dfrac{K\ell u}{r} > 100$

BRACED
FOR SIDESWAY

NOT BRACED
FOR SIDESWAY

SEE YOUR
SUPERVISOR

SHORT
COLUMNS

LONG
COLUMNS

SHORT
COLUMNS

LONG
COLUMNS

$\dfrac{K\ell u}{r} \leq 34\text{-}12\dfrac{M1}{M2}$

$\dfrac{K\ell u}{r} > 34\text{-}12\dfrac{M1}{M2}$

$\dfrac{K\ell u}{r} < 22$

$\dfrac{K\ell u}{r} \geq 22$

COMPUTE
MAGNIFICATION
FACTOR "δ"

DESIGN SECT.
USING ULT.
MOMENTS

DESIGN SECT.
USING ULT. MOM.
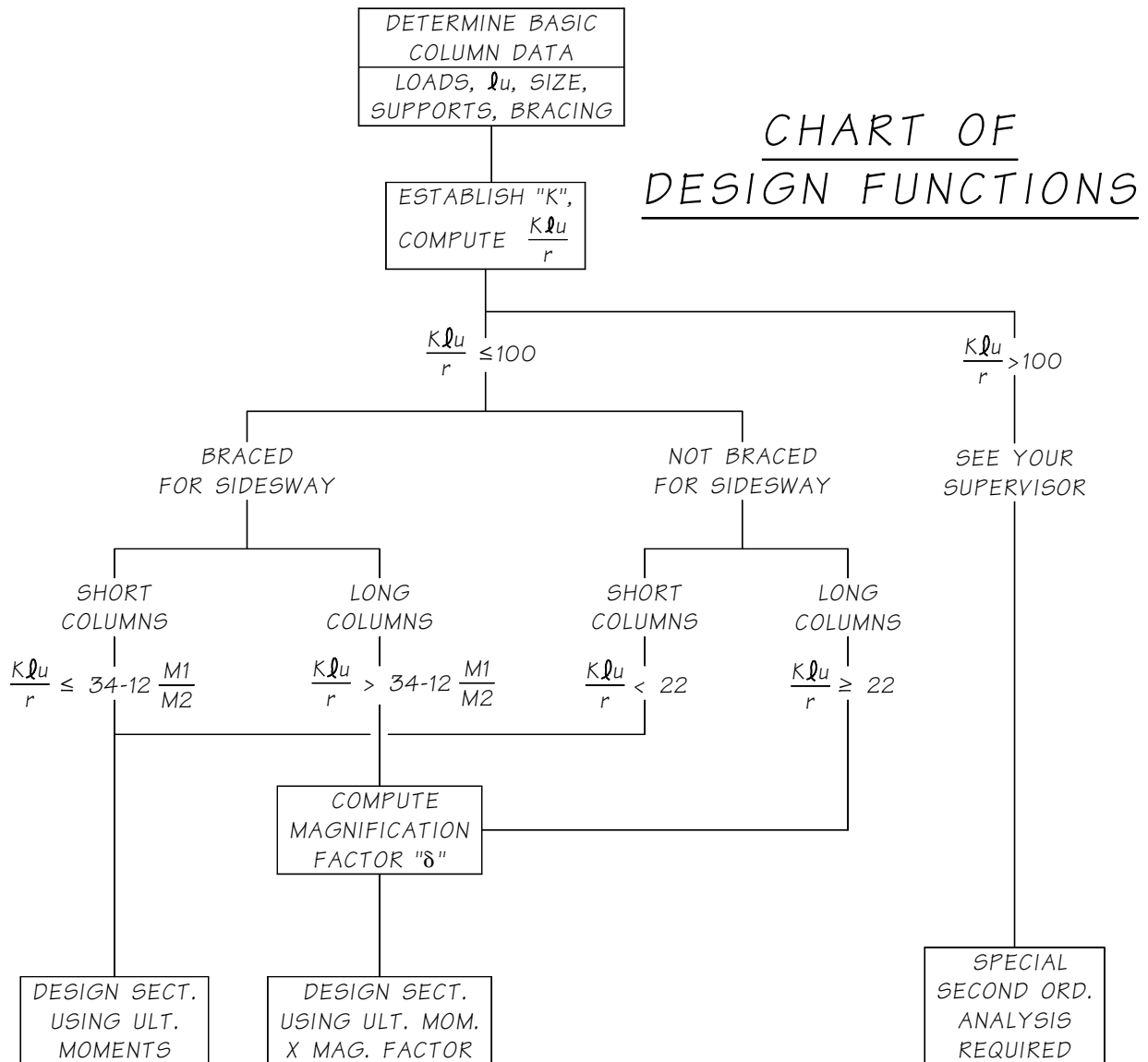X MAG. FACTOR

SPECIAL
SECOND ORD.
ANALYSIS
REQUIRED

Figure 7.3.3-1

7.3.4 Slenderness Effects

This BDM section supplements and clarifies AASHTO specifications. The goal of a slenderness analysis is to estimate the additional bending moments in the columns that are developed due to axial loads acting upon a deflected structure. Two primary analysis methods exist: the moment magnifier method and the second-order analysis. The designer must decide which method to use based upon the slenderness ratio (kLu/r) of the column(s).

Method 1:   Allowed if kLu/r < 100. BDM Section 7.3.5 discusses the approximate moment magnifier method that is generally more conservative and easier to apply.

Method 2:   Recommended by AASHTO for all situations and is mandatory for $kL_u/r > 100$. BDM Section 7.3.6 discusses a second-order structural analysis that accounts directly for the axial forces and can lead to significant economy in the final structure.

In general, tall thin columns and piles above ground (pile bents) are considered unbraced and larger short columns are considered braced.

A. Braced or Unbraced Columns

In a member with loads applied at the joints, any significant deflection "side ways" indicated the member is unbraced. The usual practice is to consider the pier columns as unbraced in the transverse direction. The superstructure engages girder stops at the abutment and resists lateral sidesway due to axial loads. However, pier lateral deflections are significant and are considered unbraced. Short spanned bridges may be an exception.

Most bridge designs provide longitudinal expansion bearings at the end piers. Intermediate columns are considered unbraced because they must resist the longitudinal loading. The only time a column is braced in the longitudinal direction is when a framed bracing member does not let the column displace more than L/1500. L is the total column length. In this case, the bracing member must be designed to take all of the horizontal forces.

7.3.5 Moment Magnification Method

The moment magnification method is described in AASHTO LRFD Article 4.5.3.2.2. The following information is required.

• Column geometry and properties: E, I, Lu, and k.
• All Strength loads obtained from conventional elastic analyses using appropriate stiffness and fixity assumptions and column under strength factor (ϕ).

Computations of effective length factors, k, and buckling loads, Pc, are not required for a second-order analysis, though they may be helpful in establishing the need for such an analysis. In general, if magnification factors computed using the AASHTO Specifications are found to exceed about 1.4, then a second-order analysis may yield substantial benefits.

7.3.6   Second-Order Analysis

A second-order analysis that includes the influence of axial loads on the deflected structure is required under certain circumstances, and may be advisable in others. It can lead to substantial economy in the final design of many structures. The designer should discuss the situation with the supervisor before proceeding with the analysis. The ACI Building Code (ACI 318 R-02, section 10.13.4.1) should be consulted when carrying out a second-order analysis.

For columns framed together, the entire frame should be analyzed as a unit. Analyzing individual columns result in overly conservative designs for some columns and non-conservative results for others. This is a result of redistribution of the lateral loads in response to the reduced stiffness of the compression members. For example, in a bridge with long, flexible columns and with short, stiff columns both integrally connected to a continuous superstructure, the stiff columns will tend to take a larger proportion of the lateral loading as additional sidesway under axial loads occurs.

A.   Design Methods for a Second-Order Analysis

The preferred method for performing a second-order analysis of an entire frame or isolated single columns is to use a nonlinear finite element program, such as GTSTRUDL, with appropriate stiffness and restraint assumptions. The factored group loads are applied to the frame, including the self-weight of the columns. The model is then analyzed using the nonlinear option available in GTSTRUDL. The final design moments are obtained directly from the analysis.

P-Δ moments are added to the applied moments using an iterative process until stability is reached. The deflections should converge within 5% of the total deflection. Analysis must include the effect of the column weight; therefore, the axial dead load must be adjusted as follows:

$P_u = P_u + \frac{1}{3}$ (factored column weight).

B.   Applying Factored Loads

For a second-order analysis, loads are applied to the structure and the analysis results in member forces and deflections. It must be recognized that a second-order analysis is non-linear and the commonly assumed principle of superposition may not be applicable. The loads applied to the structure should be the entire set of factored loads for the load group under consideration. The analysis must be repeated for each group load of interest. The problem is complicated by the fact that it is often difficult to predict in advance which load groups will govern.

For certain loadings, column moments are sensitive to the stiffness assumptions used in the analysis. For example, loads developed as a result of thermal deformations within a structure may change significantly with changes in column, beam, and foundation stiffness. Accordingly, upper and lower bounds on the stiffness should be determined and the analysis repeated using both sets to verify the governing load has been identified.

C. Member Properties

   As with a conventional linear elastic frame analysis, various assumptions and simplifications must be made concerning member stiffness, connectivity, and foundation restraint. Care must be taken to use conservative values for the slenderness analysis. Reinforcement, cracking, load duration, and their variation along the members are difficult to model while foundation restraint will be modeled using soil springs.

7.3.7   Shear Design

Shear design should follow the "Simplified Procedure for Nonprestressed Sections" in AASHTO LRFD Article 5.8.3.4.1.

7.4   Column Reinforcement

7.4.1   Reinforcing Bar Material

In accordance with Standard Specification Section 9-07.2, steel reinforcing bars for all bridge substructure elements (precast and cast-in-place) shall be ASTM A 706 only. ASTM A 706 specifications were developed for seismic applications and place limits on yield and tensile strengths. Also, chemistry is controlled to facilitate welding.

ASTM A 706 is available in sizes from #4 to #18 in straight bars and #3 to #6 in spirals.

7.4.2   Longitudinal Reinforcement Ratio

The reinforcement ratio is the steel area divided by the gross area of the section (As/Ag). The maximum reinforcement ratio shall be 0.06. However, generally the reinforcement ratio should not exceed 0.04. The minimum reinforcement ratio shall be 0.01.

If oversized columns are used for architectural reasons, the minimum reinforcement ratio of the gross section may be reduced to 0.005, provided all loads can be carried on a reduced section with similar shape and the reinforcement ratio of the reduced section is equal to or greater than 0.01. The column dimensions are to be reduced by the same ratio to obtain the similar shape. The reduced section properties are not used for modeling.

7.4.3   Longitudinal Splices

In general, column longitudinal reinforcement shall not be spliced at points of maximum moment, plastic hinge locations, or in columns less than 30 feet long between the top of footing, or shaft, and the bottom of crossbeam. The Bridge Plans must show splice location, length, and optional weld details. Standard Specification Section 6-02.3(24)F covers requirements for mechanical splices.

Column longitudinal reinforcement splices shall be staggered. For column intermediate construction joints, the shortest staggered lap bar shall project above the joint 60 bar diameters or 20 bar diameters for welded splices. Figure 7.4.3-1 shows the standard practice for staggered lap splice locations.

Splices of # 11 and smaller bars may use lap slices. When space is limited, #11 and smaller bars can use welded splices, an approved mechanical butt splice, or the top bar can be bent inward (deformed by double bending) to lie inside and parallel to the bars below. When the bar size exceeds #11, a welded splice or an approved mechanical butt splice is required. The smaller bars in the splice determine the type of splice required.

Splices should be detailed to fall within the middle one-half of the column to avoid splices in plastic hinge zones. However, in extremely tall columns where a 60-foot bar cannot reach the middle half, splices should not be closer than 30 feet from the columns ends.

OPTIONAL CONSTR. JOINT
WITH ROUGHENED
SURFACE. MAXIMUM
LENGTH OF COLUMN
CONCRETE PLACEMENT IS
30'-0". MAX. LENGTH OF
REINFORCING BAR IS
60'-0".

20 DIAMETERS
OR 24" MINIMUM

REQUIRED LAP
SPLICE LENGTH
OR 60 DIAMETERS

20 DIAMETERS
OR 24" MINIMUM

MAXIMUM OF: LARGEST
COLUMN DIMENSION,
¼ COLUMN HEIGHT,
OR 18".

MAXIMUM OF: LARGEST
COLUMN DIMENSION,
¼ COLUMN HEIGHT,
OR 18".

COLUMN

SHAFT

SHAFT REINFORCING

LAP SPLICE
#11 AND SMALLER BARS
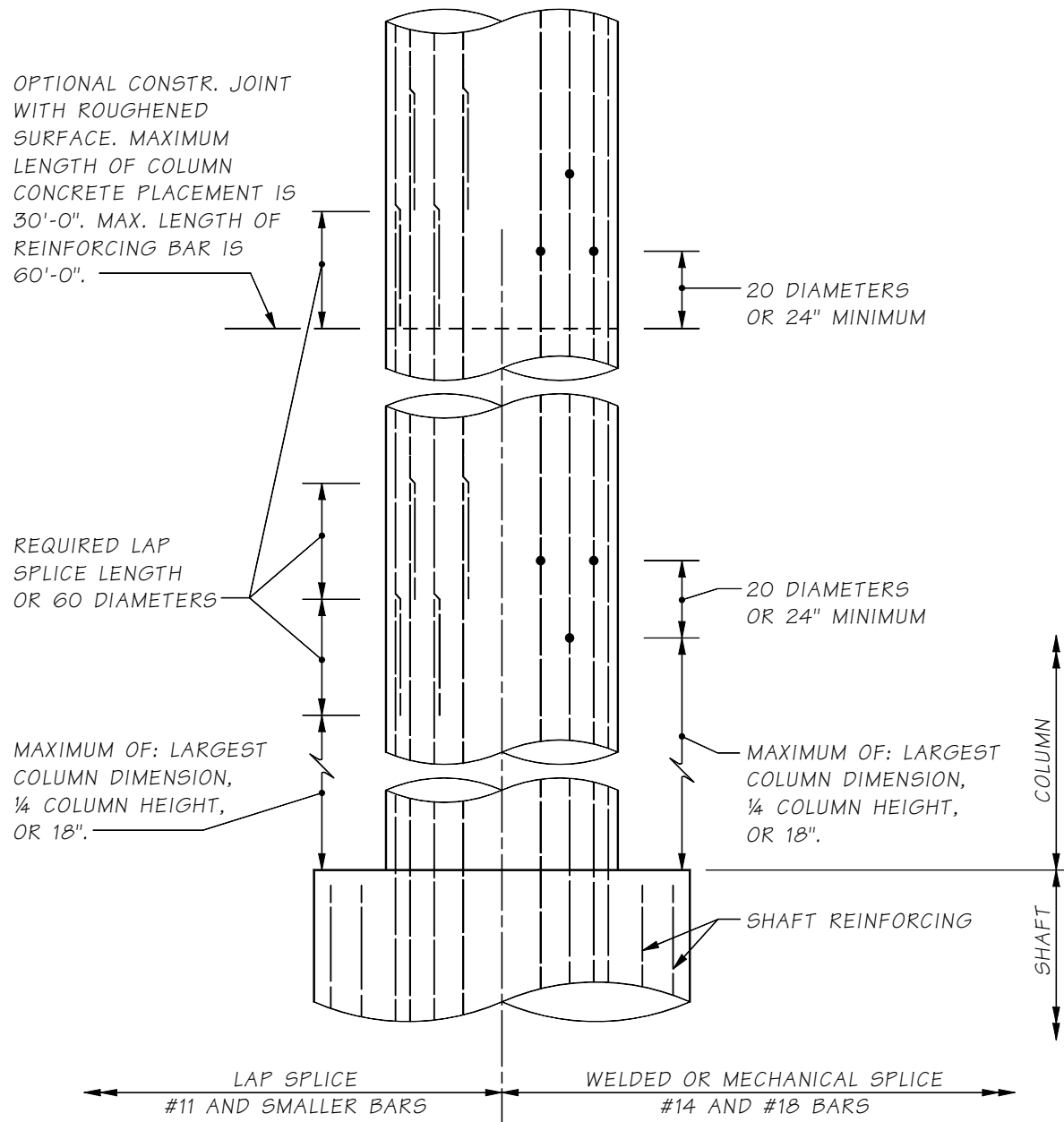
WELDED OR MECHANICAL SPLICE
#14 AND #18 BARS

Figure 7.4.3-1

7.4.4 Longitudinal Development

A. Crossbeams

Development of longitudinal reinforcement shall be in accordance with Article 8.8.4 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

A detail showing horizontal lower crossbeam reinforcement and vertical column reinforcement is preferred but not required.

B. Footings

Longitudinal reinforcement at the bottom of a column should extend into the footing and rest on the bottom mat of footing reinforcement with standard 90° hooks. In addition , development of longitudinal reinforcement shall be in accordance with Article 8.8.4 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design.
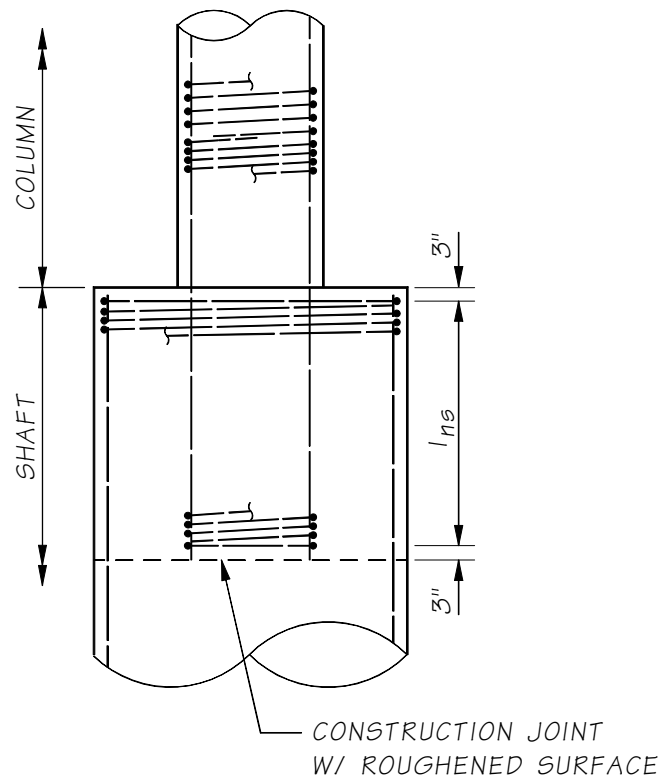
C. Drilled Shafts

Column longitudinal reinforcement in drilled shafts is typically straight. Embedment should be equal to $l_{ns} = l_s + s$ (Noncontact Lap Splices in Bridge Column-Shaft Connections, July 1997), where:

  • l_s = lap splice length required by AASHTO LRFD Article 5.11.5.3, or
  • $l_s = 1.7l_d$ (for a Class C lap splice) where l_d is the development length of the larger bar
  • s = distance between the shaft and column longitudinal reinforcement

Since l_s is a function of l_d, all applicable modification factors for development length, except one, in AASHTO LRFD Article 5.11.2 may be used when calculating l_d. The modification factor in 5.11.2 that allows ld to be decreased by the ratio of (A_s required)/(A_s provided), should not be used. Using this modification factor would imply that the reinforcement does not need to yield to carry the ultimate design load. This may be true in other areas. However, our shaft/column connections are designed to form a plastic hinge, and therefore the reinforcement should have adequate development length to allow the bars to yield.

See Figure 7.4.4-1 for an example of longitudinal development into drilled shafts.

Longitudinal Development Into Drilled Shafts
Figure 7.4.4-1

7.4.5   Transverse Reinforcement

A. General

All columns in high seismic zones shall use spiral transverse reinforcement. Columns in low seismic zones may use spirals or rectangular hoops and crossties. Figures 7.4.5-1 and 7.4.5-2 show transverse reinforcement details for rectangular columns in high and low seismic zones, respectively.
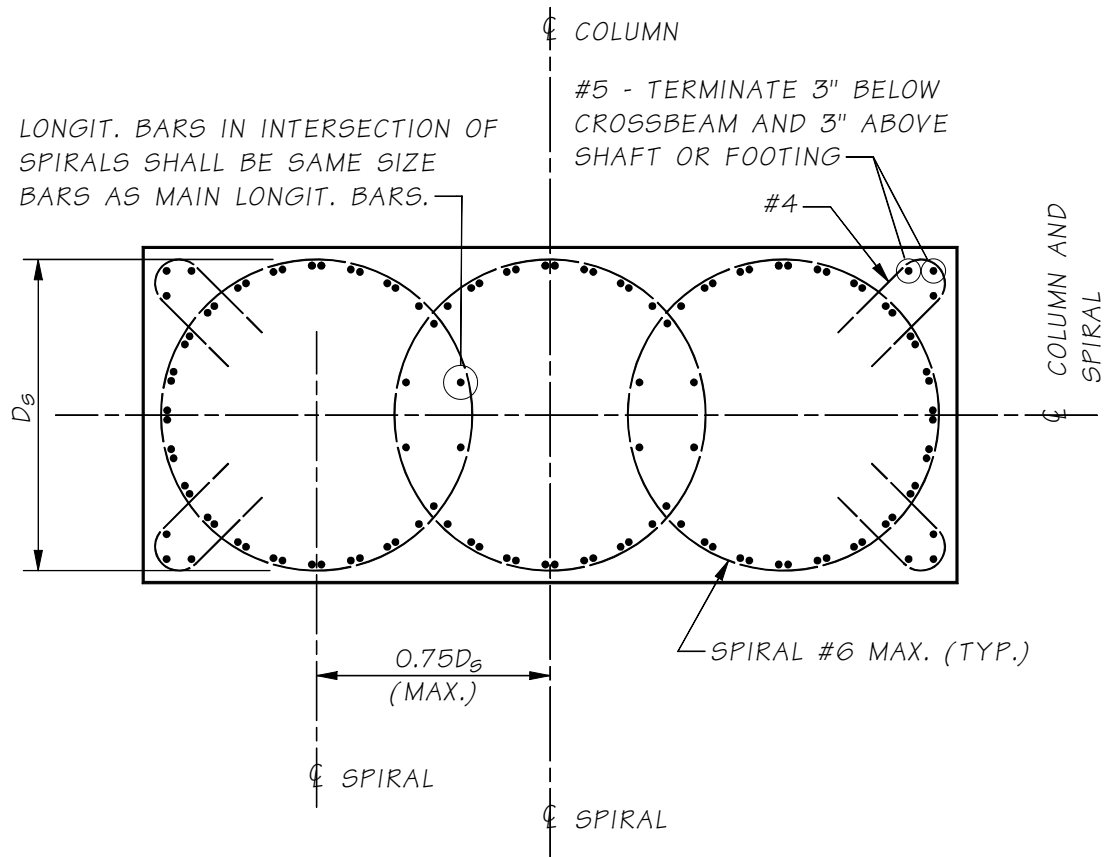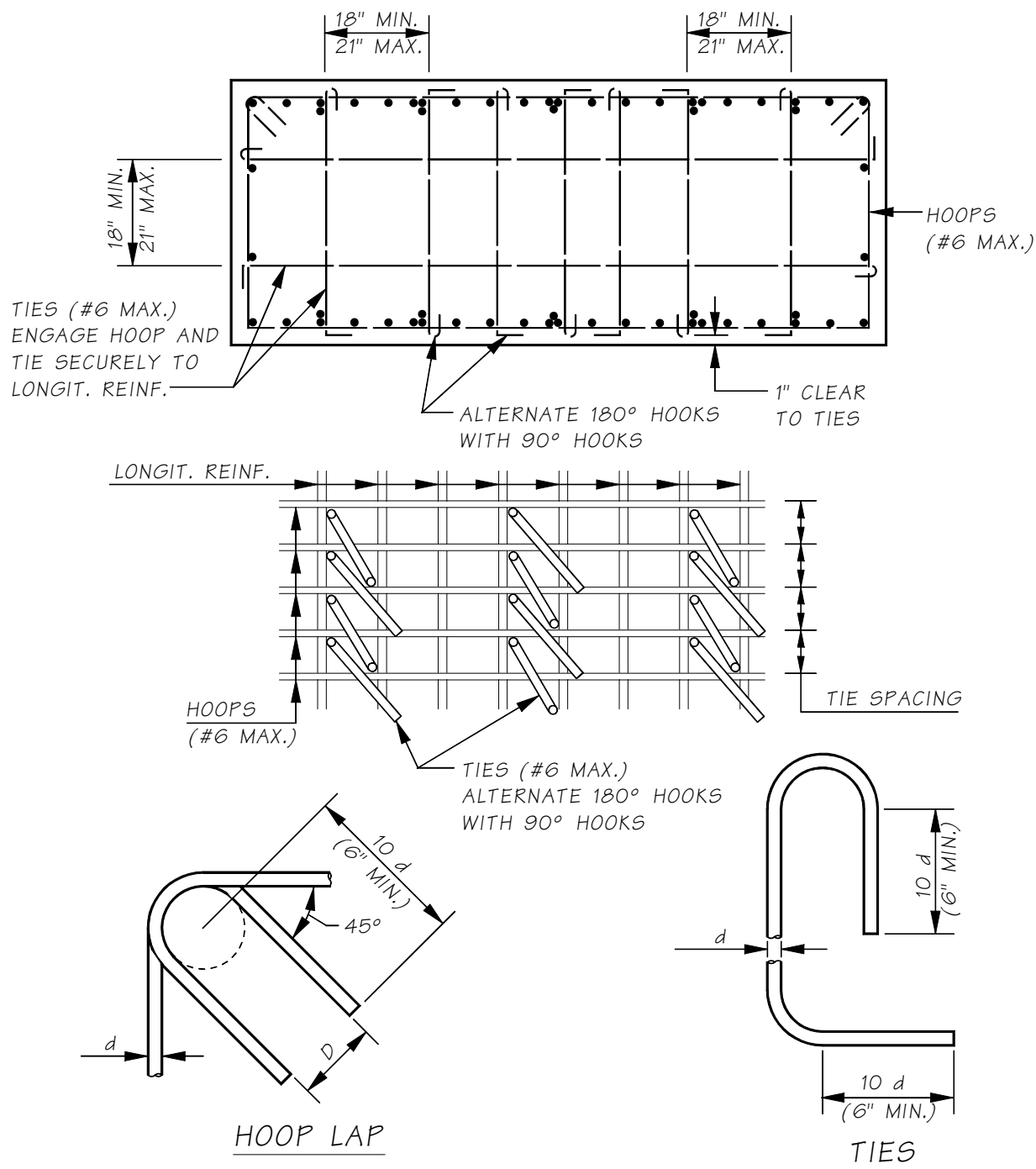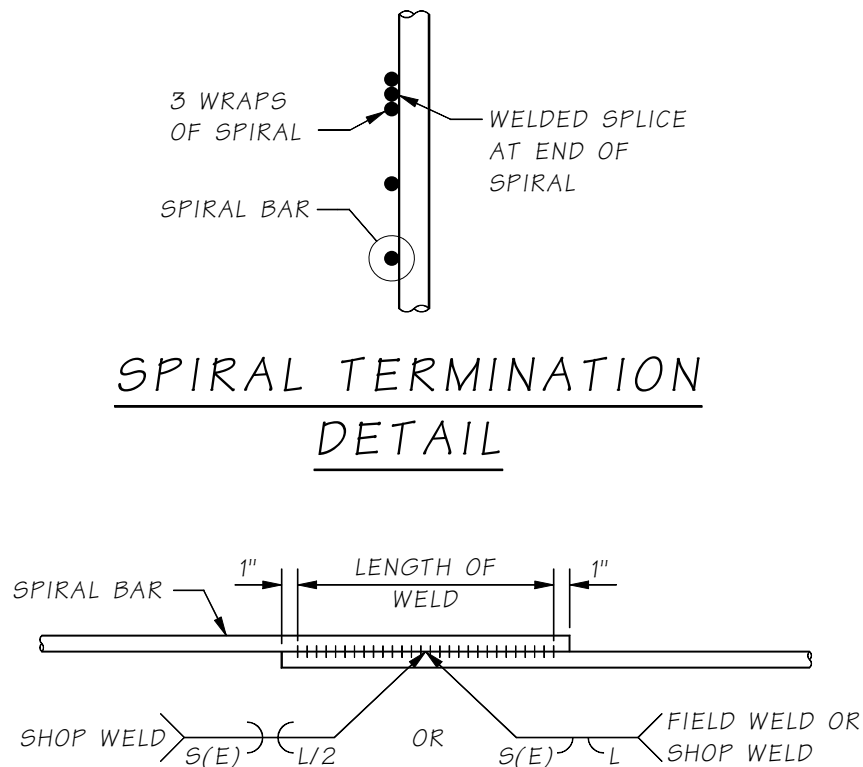

Figure 7.4.5-1

Figure 7.4.5-2

B. Spiral Splices

Only welded spiral splices are allowed. If a contractor prefers to use a lap splice, the request will be considered on a case-by-case basis. Only welded spiral splices shall be shown on the plans.

If a lap splice is allowed, only deformed bars (ASTM A 706) shall be used. Plain bars shall not be allowed for lap splices because the lap splice option has only been tested for deformed bars under seismic loads.

Although lap splices are structurally acceptable, and permissible by AASHTO, they cause construction challenges. While casting concrete, tremies get caught in the protruding hooks, making accessibility to all areas and its withdrawal cumbersome.

See Figure 7.4.5-3 for an example of a welded splice detail.

3 WRAPS OF SPIRAL

WELDED SPLICE AT END OF SPIRAL

SPIRAL BAR

SPIRAL TERMINATION DETAIL

SPIRAL BAR

1"

LENGTH OF WELD

1"

SHOP WELD   S(E)   L/2   OR   S(E)   L   FIELD WELD OR SHOP WELD

WELDED SPLICE DETAIL

WELDING SHALL MEET THE REQUIREMENTS OF STD. SPEC. 6-02.3(24)E
FOR WELD DIMENSIONS, SEE TABLE BELOW.

WELD DIMENSIONS

	WELD DIMENSIONS		
	S	E	LENGTH (L)
#4	¼	⅛	4
#5	5⁄16	3⁄16	6
#6	⅜	3⁄16	6

Welded Spiral Splice
Figure 7.4.5-3

7.4.6   Hinge Diaphragms

Hinge diaphragms of the type shown in Figure 7.4.6-1 were built on past WSDOT bridges. Typically they were used above a crossbeam or wall pier. These types of hinges are suitable when widening an existing bridge crossbeam or wall pier with this type of detail.

The area of the hinge bars in square inches is as follows:

$$As = \frac{\frac{(P_u)}{2} + \left[\frac{P_u^2}{4} + V_u^2 \right]^{1/2}}{0.85 \, F_y \, Cos \, \theta}$$

Where:

P_u is the factored axial load
V_u is the factored shear load
F_y is the reinforcing yield strength (60 ksi)
θ is the angle of the hinge bar to the vertical

The development length required for the hinge bars is 1.25 ld. All applicable modification factors for development length in AASHTO LRFD Article 5.11.2 may be used when calculating l_d. Tie and spiral spacing should conform to AASHTO LRFD confinement and shear requirements. Ties and spirals should not be spaced more than 12 inches (6 inches if longitudinal bars are bundled). Premolded joint filler should be used to assure the required rotational capacity. There should also be a shear key at the hinge bar location.

When the hinge reinforcement is bent, additional confinement reinforcing may be necessary to take the horizontal component from the bent hinge bars. The maximum spacing of confinement reinforcing for the hinge is the smaller of that required above and the following:

$$S_{max} = \frac{A_v \, F_y}{\left[\frac{P_u \, Tan \, \theta}{0.85 \, l_h} + \frac{V_s}{d} \right]}$$

Where:

A_v, V_s, and d are as defined in AASHTO Article "Notations" and 1h is the distance from the hinge to where the bend begins.
Continue this spacing one-quarter of the column width (in the plane perpendicular to the hinge) past the bend in the hinge bars.
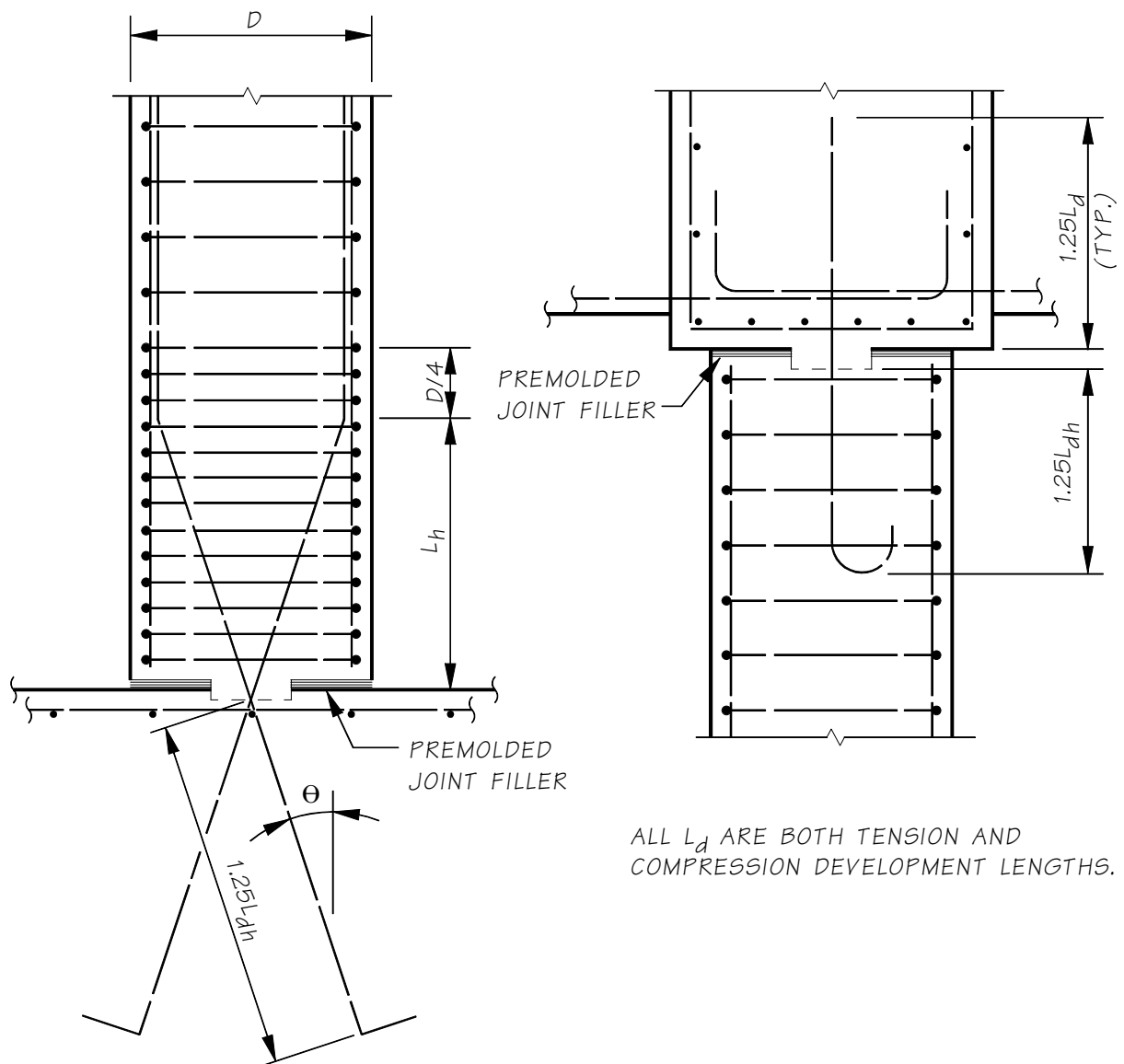
Hinge Details
Figure 7.4.6-1

7.4.7   Column Hinges

Column hinges should be detailed as shown in Figure 7.4.7-1. Details of this design can be found in "Seismic Design of Bridges Design Example No. 4" (FHWA –SA-97-009). This example is for a hinge at the bottom of a column and is based on AASTHTO Load Factor Design. New designs may use this type of connection at the top and bottom of a column but shall be modified appropriately to follow the current AASHTO Load and Resistance Factor Design Specifications, including the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

In particular the following guidelines shall be followed when designing these types of column connections:

A. The inner core, or hinge, shall not be idealized as a hinge and must be designed to resist all bending moment demands from Strength and Service Load Combinations. The inner core, or hinge, shall only be idealized and designed as a hinge for seismic loading.

B. The inner core longitudinal reinforcement and spiral shall be designed for column axial load and interface shear friction requirements.

C. The resistance factor used to design the inner core, or hinge, shall be that for compression controlled sections in AASHTO LRFD Article 5.5.4.2.1 in all seismic zones. Currently the resistance factor for compression controlled sections is 0.75.

D. The column above or below this connection shall be designed as typical column. All specifications pertaining to resistance factors shall apply.

E. Design of the non-contact lap splice between the column reinforcement and hinge reinforcement shall follow the requirements for single column/single shafts. Specifically, the spiral pitch in the column shall follow the requirements of BDM Section 7.8.2 and the development length of the hinge longitudinal reinforcement shall follow the requirements of BDM Section 7.4.4.
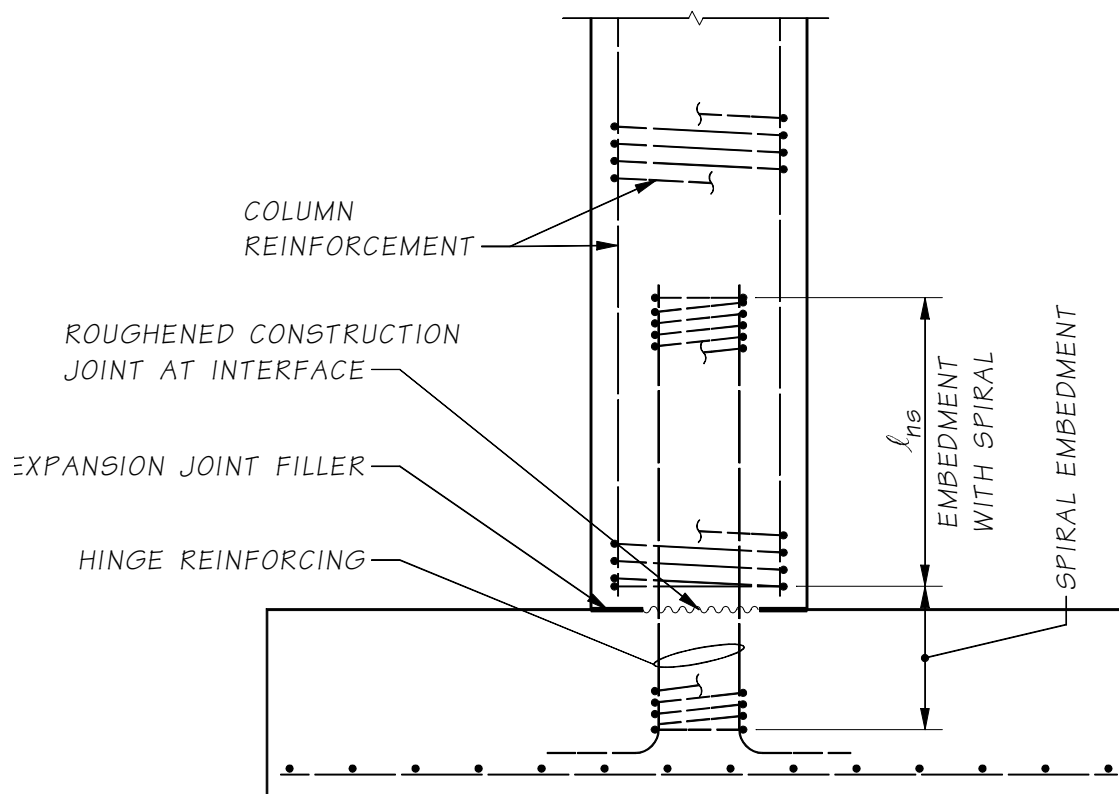
Pinned Column Base
Figure 7.4.7-1

7.5 Abutment Design and Details

7.5.1 Abutment Types

There are four abutment types described in the following section that have been used by the Bridge and Structures Office. The representative types are intended for guidance only and may be varied to suit the requirements of the bridge being designed.

A. Stub Abutments

Stub abutments are short abutments where the distance from the girder seat to top of footing is less than approximately 4 feet, see Figure 7.5.1-3. The footing and wall can be considered as a continuous inverted T-beam. The analysis of this type abutment shall include investigation into both bending and shear stresses parallel to centerline of bearing. If the superstructure is relatively deep, earth pressure combined with longitudinal forces from the superstructure may become significant.
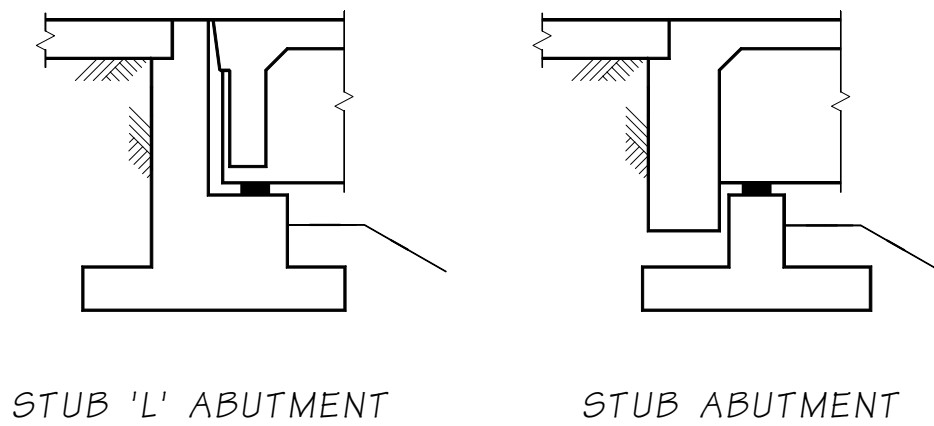
STUB 'L' ABUTMENT   STUB ABUTMENT

Figure 7.5.1-3

B. Cantilever Abutments

If the height of the wall from the bearing seat down to the bottom of the footing exceeds the clear distance between the girder bearings, the assumed 45° lines of influence from the girder reactions will overlap, and the dead load and live load from the superstructure can be assumed equally distributed over the abutment width. The design may then be carried out on a per-foot basis. The primary structural action takes place normal to the abutment, and the bending moment effect parallel to the abutment may be neglected in most cases. The wall is assumed to be a cantilever member fixed at the top of the footing and subjected to axial, shear, and bending loads see Figure 7.5.1-4.
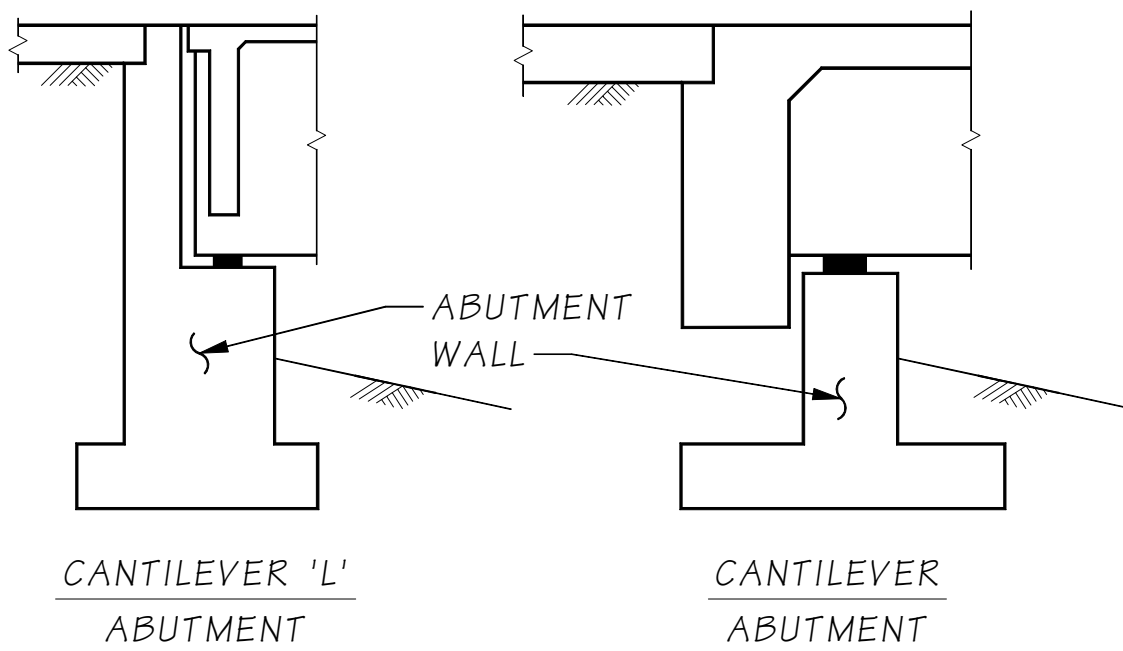
Figure 7.5.1-4

C. Spill-Through Abutments

The analysis of this type of abutment is similar to that of an intermediate pier, see Figure 7.5.1-5. The crossbeam shall be investigated for vertical loading as well as earth pressure and longitudinal effects transmitted from the superstructure. Columns shall be investigated for vertical loads combined with horizontal forces acting transversely and longitudinally. For earth pressure acting on rectangular columns, assume an effective column width equal to 1.5 times the actual column width. Short, stiff columns may require a hinge at the top or bottom to relieve excessive longitudinal moments.
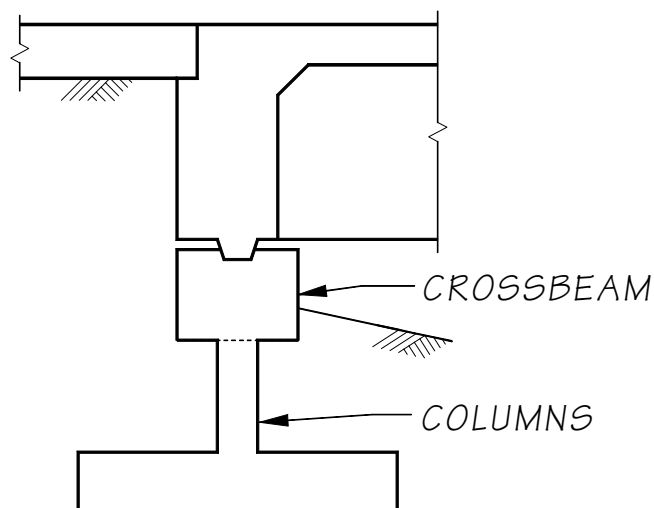
Spill-Through Abutment
Figure 7.5.1-5

D. Rigid Frame Abutments

Abutments that are part of a rigid frame are generically shown in Figure 7.5.1-6. At-Rest earth pressures (EH) will apply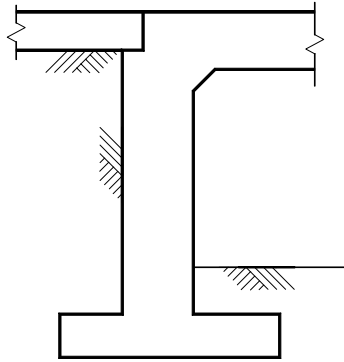 to these structures. The abutment design should include the live load impact factor from the superstructure. However, impact shall not be included in the footing design. The rigid frame itself should be considered restrained against sidesway for live load only. AASHTO Chapter 12 addresses loading and analysis of rigid frames that are buried (box culverts).

Rigid Frame Abutment
Figure 7.5.1-6

7.5.2 Embankment at Abutments

The minimum clearances for the embankment at the front face of abutments shall be as indicated on Standard Plan H-9. At the ends of the abutment, the fill may be contained with wing walls or in the case of concrete structures, placed against the exterior girders.

7.5.3 Abutment Loading

In general, bridge abutment loading shall be in accordance with AASHTO LRFD Chapter 3. The following simplifications and assumptions may be applied to the abutment design. See Section 7.5 for a force diagram of typical loads as they are applied to an abutment spread footing.

A. Dead load - DC

Approach slab dead load reaction taken as 2 kips/foot of wall applied at the pavement seat.

Active earth pressure (EH) and unit weight of backfill and toe fill (EV) will be provided in a Geotechnical Report. The toe fill should be included in the analysis for overturning if it adds to overturning.

The passive earth pressure exerted by the fill in front of the abutment is usually neglected in the design. The Geotechnical Branch should be contacted to determine if passive resistance might be considered for analysis of sliding stability. Passive resistance in front of footing is not dependable due to potential for erosion, scour, or future excavation in front of footing.

B. Live load - LL

Live load impact does not apply to the abutment. Bridge approach slab live load reaction (without IM) applied at the pavement seat may be assumed to be 4.5 kips per foot of wall for HL-93 loading, see BDM Section 10.6 for bridge approach slab design assumptions. Abutment footing live loads may be reduced (by approximately one axle) if one design truck is placed at the bridge abutment with a bridge approach slab. Adding the pavement seat reaction to the bearing reaction duplicates the axle load from two different design truck configurations.

If bridge approach slabs are not to be constructed in the project (e.g. bride approach slab details are not included in the bridge sheets of the Plans) a live load surcharge (LS) applies.

C. Earthquake Load - EQ

Superstructure loads shall be transmitted to the substructure through bearings, girder stops or restrainers. As an alternate, the superstructure may be rigidly attached substructure.

The horizontal earth pressure load (EQ_{soil}) shall be the Mononobe-Okabe (M-O) active pressure coefficient, as described in the LRFD Chapter 11, Appendix 11.1.1.1. This applies M-O as a uniform pressure to the wall with the resultant force located at 0.5H. For more information on Mononobe-Okabe and AASHTO application, see GDM Section 15.4.2.9.

Footing supported walls and abutments that are free to translate or move during a seismic event shall use Mononobe-Okabe soil pressure. The vertical acceleration, k_v, shall be set equal to 0. This also applies to portions substructure isolated from the superstructure by bearings.

Pile or shaft walls and abutments that are not free to translate or move during a seismic event shall use a horizontal acceleration of 1.5 times peak ground acceleration. The vertical acceleration shall be set equal to 0. See GDM Section 15.4.2.7 for descriptions of flexible and non-yielding walls.

Seismic inertial force of the substructure (EQ_{abut}) is the horizontal acceleration coefficient times the weight of the abutment (including footing). This force acts horizontally in the same direction as the earth pressure, at the mass centroid of the abutment. Seismic inertia force is only applied for stability and sliding analysis. EQ_{abut} shall not be used to determine the reinforcement required in the abutment.

The load factor for all EQ induced loads shall be 1.0, including M-O earth pressure loads.

D. Bearing Forces – TG   Strength and Extreme Event II

For strength design, the bearing shear forces should be based on ½ of the seasonal temperature change. This force is applied in the direction that causes the worst case loading.

For extreme event II, calculate the maximum friction force (when the bearing slips) and apply in the direction that causes the worst case loading.

7.5.4   WSDOT Temporary Construction Load Cases

A. Case 1: Superstructure Built after Backfill at Abutment

If the superstructure is to be built after the backfill is placed at the abutments, the resulting temporary loading would be the maximum horizontal force with the minimum vertical force. During the abutment design, a load case shall be considered to check the stability and sliding of abutments after placing backfill but prior to superstructure placement. This load case is intended as a check for a temporary construction stage, and not meant to be a controlling load case that would govern the final design of the abutment and footing. This loading will generally determine the tensile reinforcement in the top of the footing heel.

If this load case check is found to be satisfactory, a note shall be added to the general notes in the contract plans and the contactor will not be required to make a submittal requesting approval for early backfill placement. This load case shall include a 2-foot deep soil surcharge for the backfill placement equipment (LS) as covered by the WSDOT Standard Specification Section 2-03.3(14)I.

B. Case 2: Wingwall Overturning

It is usually advantageous in sizing the footing to release the falsework from under the wing walls after some portion of the superstructure load is applied to the abutment. A note can cover this item, when applicable, in the sequence of construction on the plans.

7.5.5   Abutment Bearings and Girder Stops

All structures shall be provided with some means of restraint against lateral displacement at the abutments due to earthquake, temperature and shrinkage, wind, earth pressure, etc. Such restraints may be in the form of concrete hinges, concrete girder stops with or without vertical elastomeric pads, or pintles in metal bearings. Other solutions are possible. Article "Connection Design Forces" of the Guide Specifications for Seismic Design of Highway Bridges describe longitudinal linkage force and hold-down devices required.

All prestressed girder bridges in Western Washington (within and west of the Cascade mountain range) shall have girder stops between all girders at abutments and intermediate piers. This policy is based on fact that the February 28, 2001 Nisqually earthquake caused significant damage to girder stops at bridges where girder stops were not provided between all girders. In cases where girder stops were cast prior to placement of girders and the 3" grout was placed after setting the girders, the 3" grout pads were severely damaged and were displaced from their original position.

A. Abutment Bearings

The longitudinal forces from the superstructure are normally transferred to the abutments through the bearings. The calculated longitudinal movement shall be used to determine the shear force developed by the bearing pads at the abutments. The Modulus of Elasticity of Neoprene at 70°F (21°C) shall be used for determining the shear force. However, the force transmitted through a bearing pad shall be limited to that which causes the bearing pad to slip. Normally, the maximum load transferred through a teflon sliding bearing is 6 percent and through an elastomeric bearing pad is 20 percent of the dead load reaction of the superstructure. For Extreme Event I, assume the end diaphragm is in contact with abutment wall and no load transfer through the bearings. The bearing force shall not be added to seismic earth pressure forces.

When the force transmitted through the bearing pads is very large, the designer should consider increasing the bearing pad thickness, using TFE sliding bearings and/or utilizing the flexibility of the abutment as a means of reducing the horizontal design force. When the flexibility of the abutment is considered, it is intended that a simple approximation of the abutment deformation be made.

B. Bearing Seats

The bearing seats shall be wide enough to accommodate the size of the bearings used with a minimum edge dimension of 3 in. and satisfy the requirements of LRFD Section 4.7.4.4. On L abutments, the bearing seat should be sloped away from the bearings to prevent a build up or pocket of water at the bearings. The superelevation and profile grade of the structure should be considered for drainage protection. Normally, a ¼ in. drop across the width of the bearing seat is sufficient.

C. Girder Stop Bearings

For skewed structures with earth pressure against the end diaphragm (see Figure 9.3.2-4), the performance of girder stop bearings shall be investigated at Service Limit State. These bearings are placed vertically against the girder stop to transfer the skew component of the earth pressure to the abutment without restricting the movement of the superstructure in the direction parallel to centerline. In some cases bearing assemblies containing sliding surfaces may be necessary to accommodate large superstructure movements.

D. Girder Stop Design

Some type of transverse girder stop is required for all abutments in order to transfer earthquake load from the superstructure to the abutment. The girder stop shall be designed at the Extreme Limit State for the earthquake loading, any transverse earth pressure from skewed abutments, etc. Girder stops are designed using shear friction theory. The possibility of torsion combined with horizontal shear when the load does not pass through the centroid of the girder stop shall also be investigated.

E. Girder Stop Detail

The detail shown in Figure 7.5.5-1 may be used for bridges with no skew. Prestressed girders should be placed in final position before girder stops are cast to eliminate alignment conflicts between prestressed girders and girder stops. All girder stops should provide ⅛ in clearance between the prestressed girder flange and the girder stop.
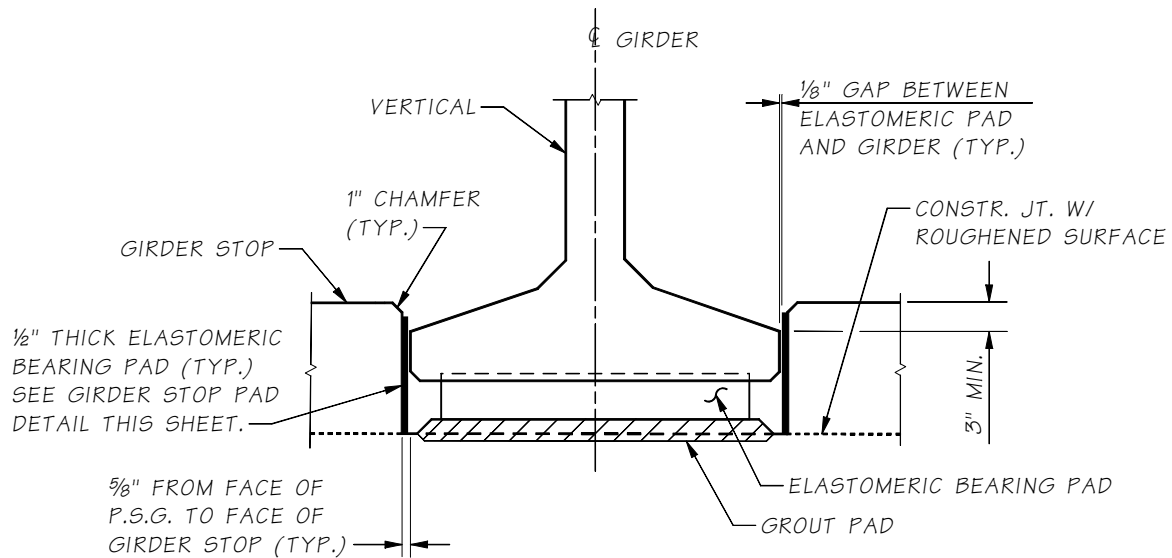
Girder Stop Details
Figure 7.5.5-1

7.5.6 Abutment Expansion Joints

For structures without expansion joints, the earth pressure against the end diaphragm is transmitted through the superstructure. The compressibility of the expansion joint shall be considered in the design of the abutment for earthquake, temperature, and shrinkage when these forces increase the design load.

7.5.7 Open Joint Details

Vertical expansion joints extending from the top of footings to the top of the abutment are usually required between abutments and adjacent retaining walls to handle anticipated movements. The expansion joint is normally filled with premolded joint filler which is not water tight. There may be circumstances when this joint must be water tight; ⅛ butyl rubber may be used to cover the joint. The open joint in the barrier should contain a compression seal to create a watertight joint. Figure 7.5.7-1 shows typical details that may be used. Aesthetic considerations may require that vertical expansion joints between abutments and retaining walls be omitted. This is generally possible if the retaining wall is less than 60 feet long.
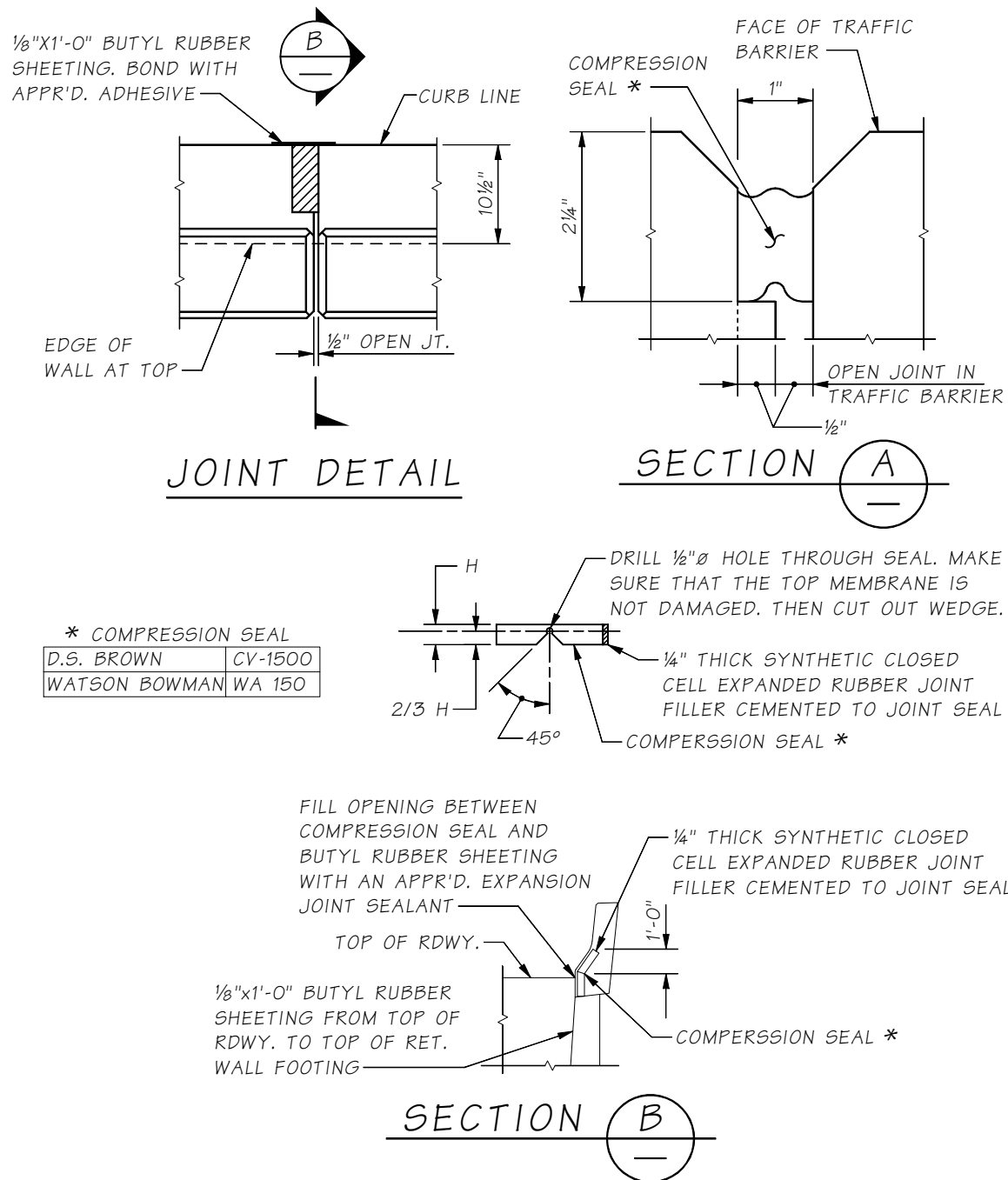
Open Joint Details between Abutment and Retaining Walls
Figure 7.5.7-1

The footing beneath the joint may be monolithic or cast with a construction joint. In addition, dowel bars may be located across the footing joint parallel to the wall elements to guard against differential settlement or deflection.

On abutments with the end diaphragm cast on the superstructure, the open joints must be protected from the fill spilling through the joint. Normally butyl rubber is used to seal the openings. See the end diaphragm details in the Appendices in BDM Chapter 5 for details.

7.5.8   Construction Joints

To simplify construction, vertical construction joints are often necessary, particularly between the abutment and adjacent wing walls. Construction joints should also be provided between the footing and the stem of the wall. Shear keys shall be provided at construction joints between the footing and the stem, at vertical construction joints or at any construction joint that requires shear transfer. The Standard Specifications cover the size and placement of shear keys. The location of such joints shall be detailed on the plans. Construction joints with roughened surface can be used at locations (except where needed for shear transfer) to simplify construction. These should be shown on the plans and labeled "Construction Joint With Roughened Surface." When construction joints are located in the middle of the abutment wall, a pour strip should be used for a clean joint between pours. Details of the pour strip should be shown in the plans.
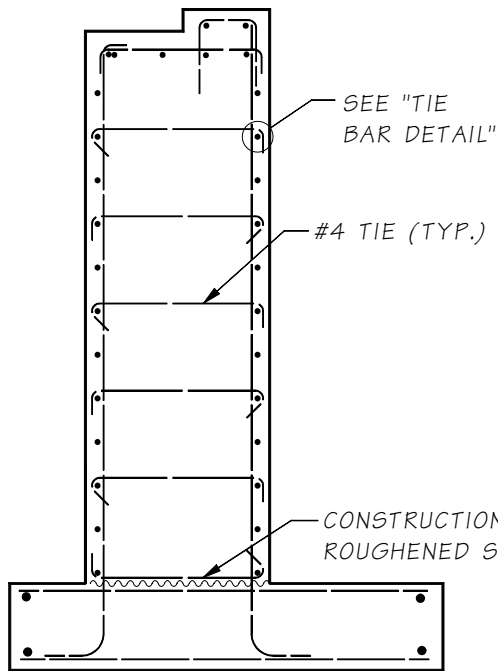
7.5.9   Abutment Wall Design

When the primary structural action is parallel to the superstructure or normal to the abutment face, the wall shall be treated as a column subjected to combined axial load and bending moment. Compressive reinforcement need not be included in the design of cantilever walls, but the possibility of bending moment in the direction of the span as well as towards the backfill shall be considered. A portion of the vertical bars may be cut off where they are no longer needed for stress.

A. In general, horizontal reinforcement should be placed outside of vertical reinforcement to facilitate easier placement of reinforcement.

B. Shrinkage and Temperature Reinforcement

   The AASHTO requires a minimum temperature and shrinkage steel of 0.125 sq. in. per foot of wall. This is not sufficient to limit shrinkage cracks in thick walls. A more appropriate minimum temperature and shrinkage steel is taken from the ACI-83, minimum area of reinforcing steel per foot of the wall, in both directions on each face of the wall, shall be 0.011 times the thickness of the wall (in inches), spaced at 12 inches. On abutments that are longer than 60 feet, consideration should be given to have vertical construction joints to minimize shrinkage cracks.

   The minimum cross tie reinforcement in the abutment wall is as follows. #4 tie bars with 180 degree hooks, spaced at approximately 2 feet center to center vertically and at approximately 4 feet center to center horizontally shall be furnished throughout the abutment stem in all but stub abutments, see Figure 7.5.9-1.

SEE "TIE
BAR DETAIL"

#4 TIE (TYP.)

CONSTRUCTION JOINT WITH
ROUGHENED SURFACE

TYPICAL SECTION

1'-6" MAX. BASED ON
HORIZ. BAR SPACING

#4 TIE (TYP.)

2'-0" MAX. BASED ON
VERTICAL BAR SPACING

TIE BAR SPACING DETAIL

#4 TIES WITH 1'-0" MIN. LAP SPLICE.
SEE "TIE BAR SPACING DETAIL"

TIE EACH END OF LAP
SPLICE WITH WIRE

1" MIN.
CLR.

135° BEND

EXTERIOR
FACE
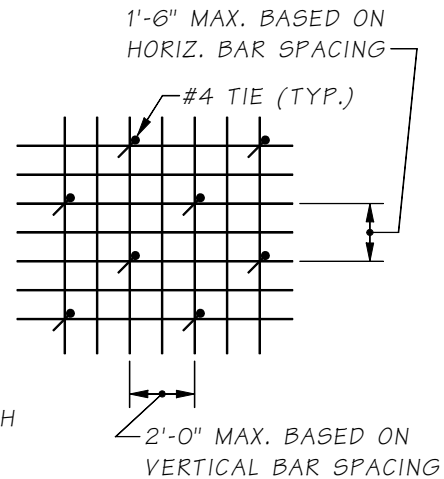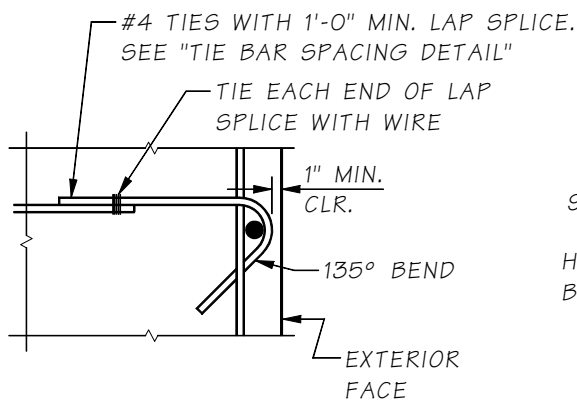
ALTERNATE TIE BAR DETAIL

CONSTANT OR VARIABLE WIDTH SECTION

135° BEND

90° BEND

HORIZONTAL
BAR (TYP.)

VERTICAL
BAR (TYP.)

1" MIN.
CLR.

#4 TIE
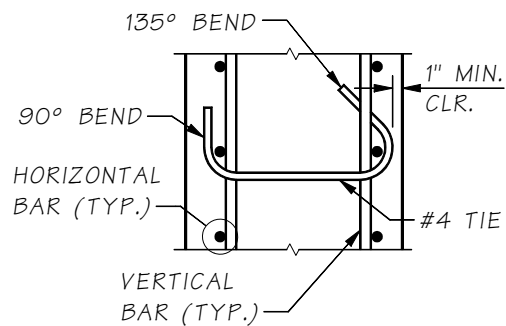
TIE BAR DETAIL

CONSTANT WIDTH SECTION

Cross Tie Details
Figure 7.5.9-1

7.5.10   Drainage and Backfilling

3" diameter weep holes shall be provided in all bridge abutment walls. These shall be located 6 inches above the final ground line at about 12 feet on centers. In cases where the vertical distance between the top of the footing and the bearing seat is greater than 10 feet, additional weep holes shall be provided 6 inches above the top of the footing. No weep holes are necessary in cantilever wing walls where a wall footing is not used.

The details for gravel backfill for wall, underdrain pipe and backfill for drain shall be indicated on the plans. The gravel backfill for wall shall be provided behind all bridge abutments. The underdrain pipe and gravel backfill for drain shall be provided behind all bridge abutments except abutments on fills with a stem wall height of 5 feet or less. When retaining walls with footings are attached to the abutment, a blockout may be required for the underdrain pipe outfall. Cooperation between Bridge and Structures Office and the Design PE Office as to the drainage requirements is needed to guarantee proper blockout locations.
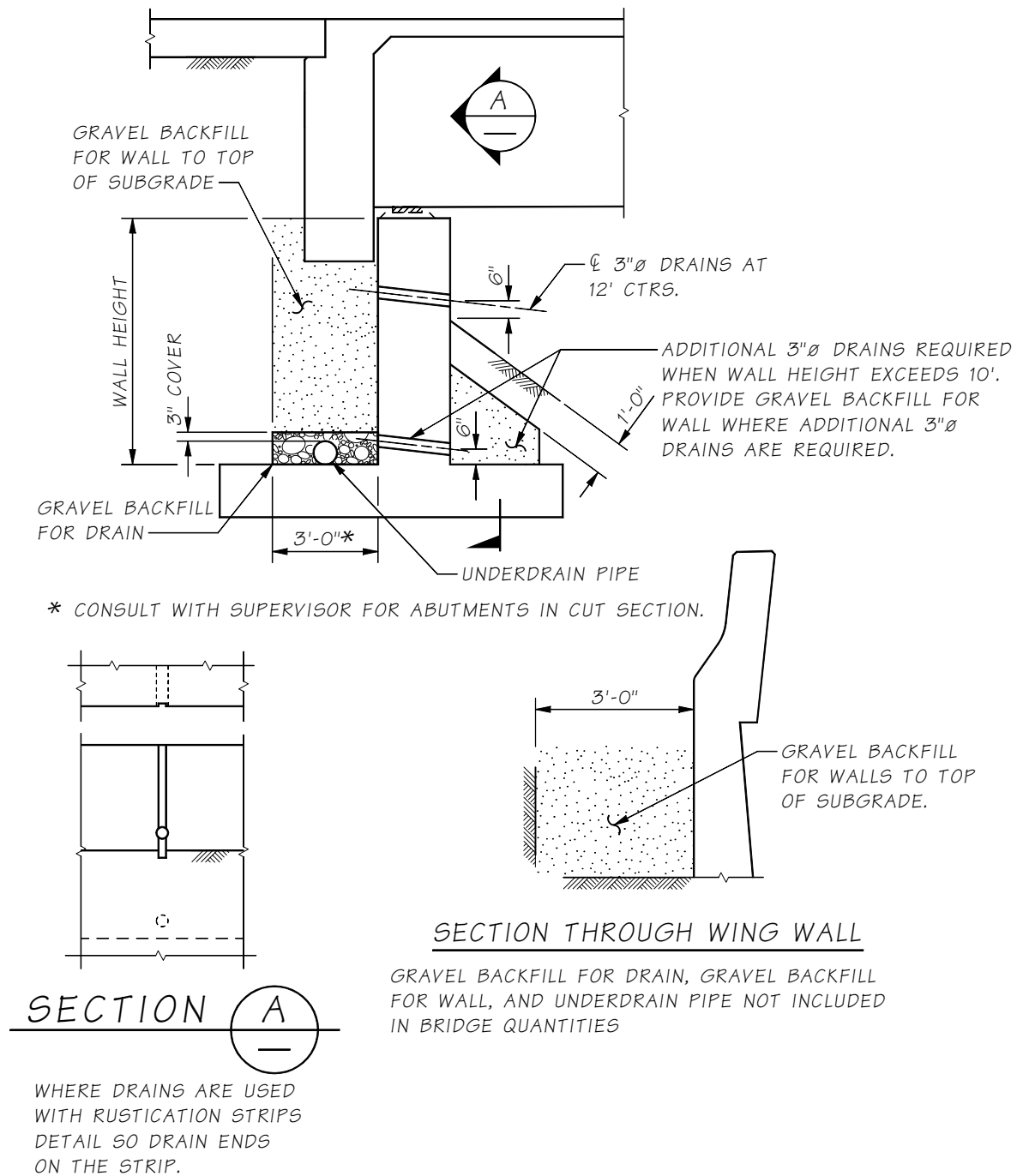
Underdrain pipe and gravel backfill for drain are not necessary behind cantilever wing walls. A 3-foot thickness of gravel backfill for wall behind the cantilever wing walls shall be shown in the plans.

The backfill for wall, underdrain pipe and gravel backfill for drain are not included in bridge quantities, the size of the underdrain pipe should not be shown on the bridge plans, as this is a Design PE Office design item and is subject to change during the design phase. Figure 7.5.10-1 illustrates backfill details.

GRAVEL BACKFILL FOR WALL TO TOP OF SUBGRADE

WALL HEIGHT

3" COVER

GRAVEL BACKFILL FOR DRAIN

3'-0"*

UNDERDRAIN PIPE

⊄ 3"ø DRAINS AT 12' CTRS.

6"

ADDITIONAL 3"ø DRAINS REQUIRED WHEN WALL HEIGHT EXCEEDS 10'. PROVIDE GRAVEL BACKFILL FOR WALL WHERE ADDITIONAL 3"ø DRAINS ARE REQUIRED.

6"

1'-0"

* CONSULT WITH SUPERVISOR FOR ABUTMENTS IN CUT SECTION.

SECTION Ⓐ

WHERE DRAINS ARE USED WITH RUSTICATION STRIPS DETAIL SO DRAIN ENDS ON THE STRIP.

3'-0"

GRAVEL BACKFILL FOR WALLS TO TOP OF SUBGRADE.

SECTION THROUGH WING WALL

GRAVEL BACKFILL FOR DRAIN, GRAVEL BACKFILL FOR WALL, AND UNDERDRAIN PIPE NOT INCLUDED IN BRIDGE QUANTITIES

Drainage and Backfill Details
Figure 7.5.10-1

7.6   Wing/Curtain Wall at Abutments

Particular attention should be given to the horizontal reinforcing steel required at fixed corners between abutment and wing/curtain walls. Since wall deflections are zero near the abutment, curtain walls and cantilever wing walls shall assume an At-Rest soil pressure. This increased loading can normally be reduced to an Active soil pressure at a distance (from the corner), equal to the average height of the wall under design. At this distance, the wall deflections are assumed large enough to allow the active state soil pressures to be developed. For the typical abutment, wingwall moments may be assumed to distribute stress to the outer 10 foot portion of the abutment wall. See Geotechnical Design Manual (GDM) Section 15.4.2.7, "Active, Passive, and At-Rest Pressures".

7.6.1   Traffic Barrier Loads

Traffic barriers should be rigidly attached to a bridge approach slab that is cantilevered over the top of a wing/curtain wall or Structural Earth wall. The barrier collision load is applied directly to the bridge approach slab. The yield line theory as specified in AASHTO LRFD Specifications article A13.3 is primarily for traffic barrier on bridge deck slabs and may not be applicable to traffic barrier on less rigid supports, such as retaining walls.

7.6.2   Wingwall Design

The following wingwall design items should be addressed in the Plans.

A. For Strength Design of wingwalls, vertical loads and moments may be distributed over 10 feet of the abutment wall and footing.

B. Footing thickness shall be not less than 1 foot 6 inches.

C. Exterior girder top flanges should be located (at the least) inside the curb line at the end pier.

D. For skewed bridges, modify the details on the traffic barrier and approach slab sheet so the expansion joint detailing agree. List appropriate manufacturers and model numbers for the expansion joint system. Generally, a 1 inch expansion joint with a 1inch open joint in the barrier is shown in the Plans, unless the bridge expansion joint design dictates otherwise.

7.6.3   Wingwall Detailing

All wingwall reinforcement should be a vertical grid and not follow a tapered bottom of wall. This allows for the steel to be placed in two layers that fits better with abutment reinforcing. Existing MicroGDS wingwall sheets conform to the LFD specifications. For consistency in design with the other bridge components, these wingwalls sheets must be re-designed in accordance with the requirements of the AASHTO LRFD Specifications.

7.7   Footing Design

7.7.1   General Footing Criteria

The provisions given in this section pertain to both spread footings and pile supported footings.

A. Minimum Cover and Footing Depth

The Geotechnical Report may specify a minimum footing depth in order to assure adequate bearing pressure. Stream crossings may require additional cover depth as protection against scour. The HQ Hydraulic Section should be consulted on this matter. Footings set too low result in large increases in cost. The end slope on the bridge approach fill is usually set at the preliminary plan stage but affects the depth of footings placed in the fill. Figure 7.7.1-1 illustrates footing criteria when setting footing elevations.
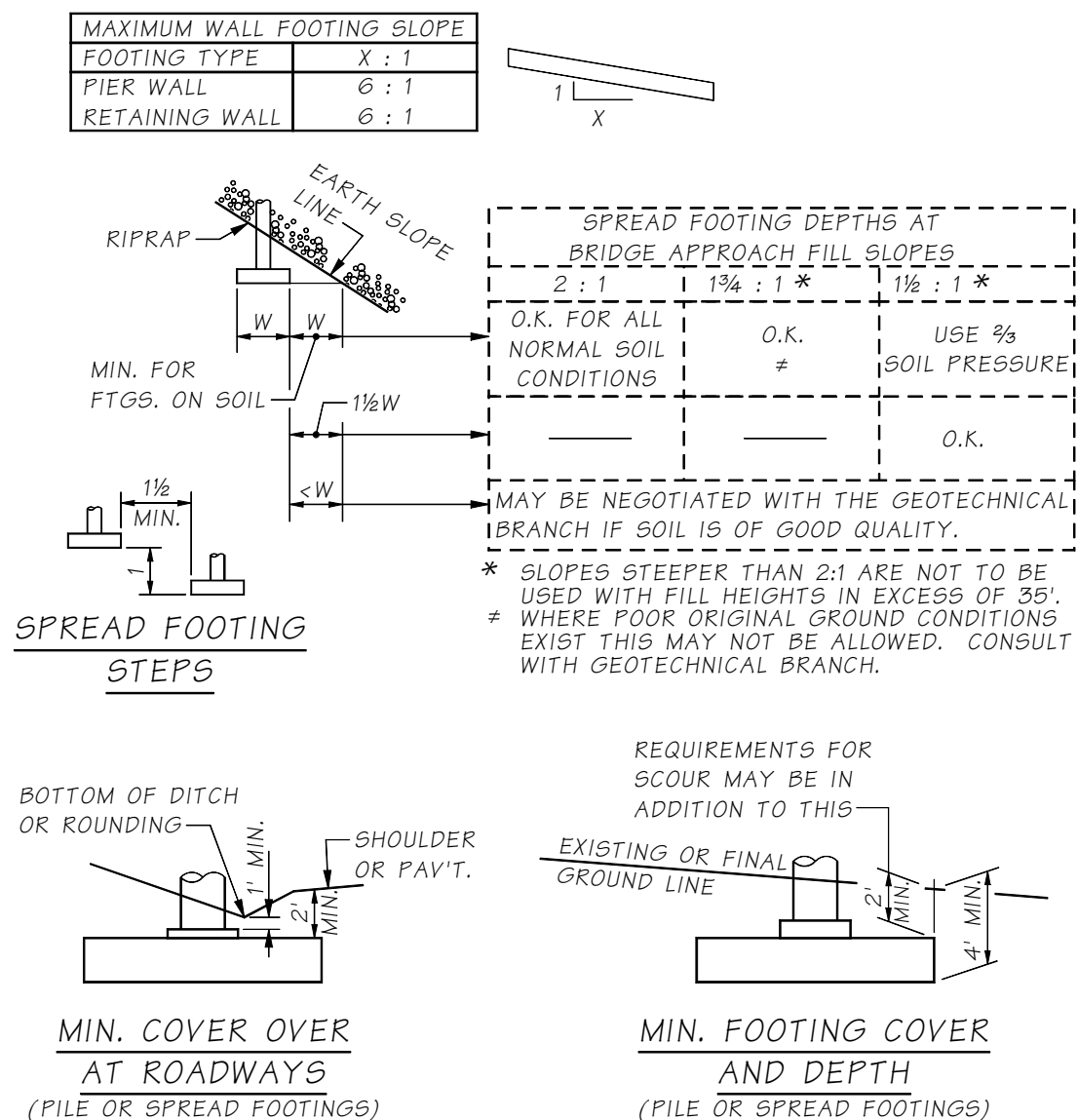

Guidelines for Footing Cover and Depth
Figure 7.7.1-1

B. Pedestals

A pedestal is sometimes used as an extension of the footing in order to provide additional depth for shear near the column. Its purpose is to provide adequate structural depth while saving concrete. For proportions of pedestals, see Figure 7.7.1-2. Since additional forming is required to construct pedestals, careful thought must be given to the trade off between the cost of the extra forming involved and the cost of additional footing concrete. Also, additional foundation depth may be needed for footing cover. Whenever a pedestal is used, the plans shall note that a construction joint will be permitted between the pedestal and the footing. This construction joint should be indicated as a construction joint with roughened surface.
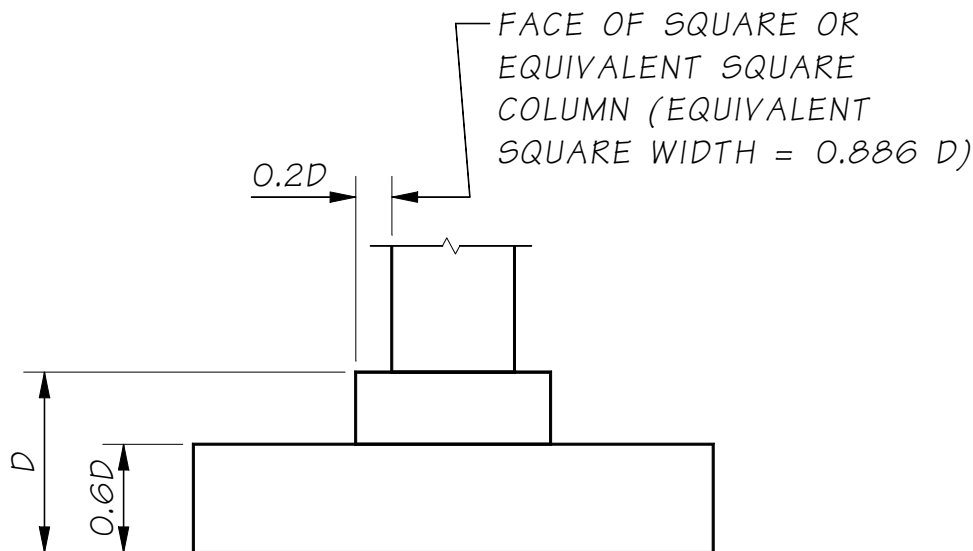
Pedestal Dimensions
Figure 7.7.1-2

7.7.2   *Loads and Load Factors*

The following Table 7.7.2-1 is a general application of minimum and maximum load factors as they apply to a generic footing design. Footing design must select the maximum or minimum Load Factors for various modes of failure for the Strength and Extreme Event Limit States.

The dead load includes the load due to structural components and non-structural attachments (DC), and the dead load of wearing surfaces and utilities (DW). The live load (LL) does not include vehicular dynamic load allowance (IM).

Designers are to note, if column design uses magnified moments, then footing design must use magnified column moments.

Sliding and Overturning, e_o	Bearing Stress (e_c, s_v)
$LL_{min} = 0$	LL_{max}
DC_{max}, DW_{max} for causing forces, DC_{max}, DW_{max} for causing forces,	DC_{min}, DW_{min} for resisting forces DC_{min}, DW_{min} for resisting forces
EV_{min}	EV_{max}
EH_{max}	EH_{max}
LS	LS

Load Factors
Table 7.7.2-1

7.7.3   Geotechnical Report Summary

The Geotechnical Branch will evaluate overall bridge site stability. Slope stability normally applies to steep embankments at the abutment. If stability is in question, a maximum service limit state load will be specified in the report. Bridge design will determine the maximum total service load applied to the embankment. The total load must be less than the load specified in the Geotechnical Report.

Based on the foundations required in the Preliminary Plan and structural information available at this stage, the Report provides the following geotechnical engineering results. For all design limit states, the total factored footing load must be less than factored resistance.

A. Plan Detailing

The Bridge Plans shall include the nominal bearing capacity in the General Notes as shown in Figure 7.7.3-1. This information is included in the Plans for future reference by the Bridge and Structures Office.

THE NOMINAL BEARING CAPACITY OF THE SPREAD
FOOTINGS SHALL BE TAKEN AS, IN KSF:

PIER NO.	SERVICE-I LIMIT STATE	STRENGTH AND EXTREME EVENT-I LIMIT STATES
1	====	====
2	====	====

Figure 7.7.3-1

B. Bearing Capacity - Service, Strength and Extreme Limit States

The unfactored bearing capacity (q_n) may be increased or reduced based on previous experience for the given soils. The Geotechnical Report will contain the following information:

- Unfactored bearing capacity (q_n) for anticipated effective footing widths, which is the same for the strength and extreme event limit states
- Resistance factor for strength limit state (ϕ_b).
- Resistance factor for the extreme event limit state (ϕ_b) is 1.0
- Service bearing capacity (q_{ser}) and amount of assumed settlement
- Embedment depth requirements or footing elevations to obtain the recommended q_n

C. Sliding Capacity - Strength and Extreme Limit States

The Geotechnical Report will contain the following information to determine earth loads and the factored sliding resistance (Q_R). $Q_R = \phi \quad Q_n = \phi$

- Resistance factor for strength limit state (ϕ_τ)
- Soil parameters ϕ_{soil}, K_a, and γ for calculating Q_τ and active force (EH) behind abutment footings
- If passive earth pressure (Q_{ep}) is allowed at a footing,
- Soil parameters of ϕ_{soil}, K_p, γ and depth of soil in front of footing
- Resistance factor ϕ_{ep} for strength
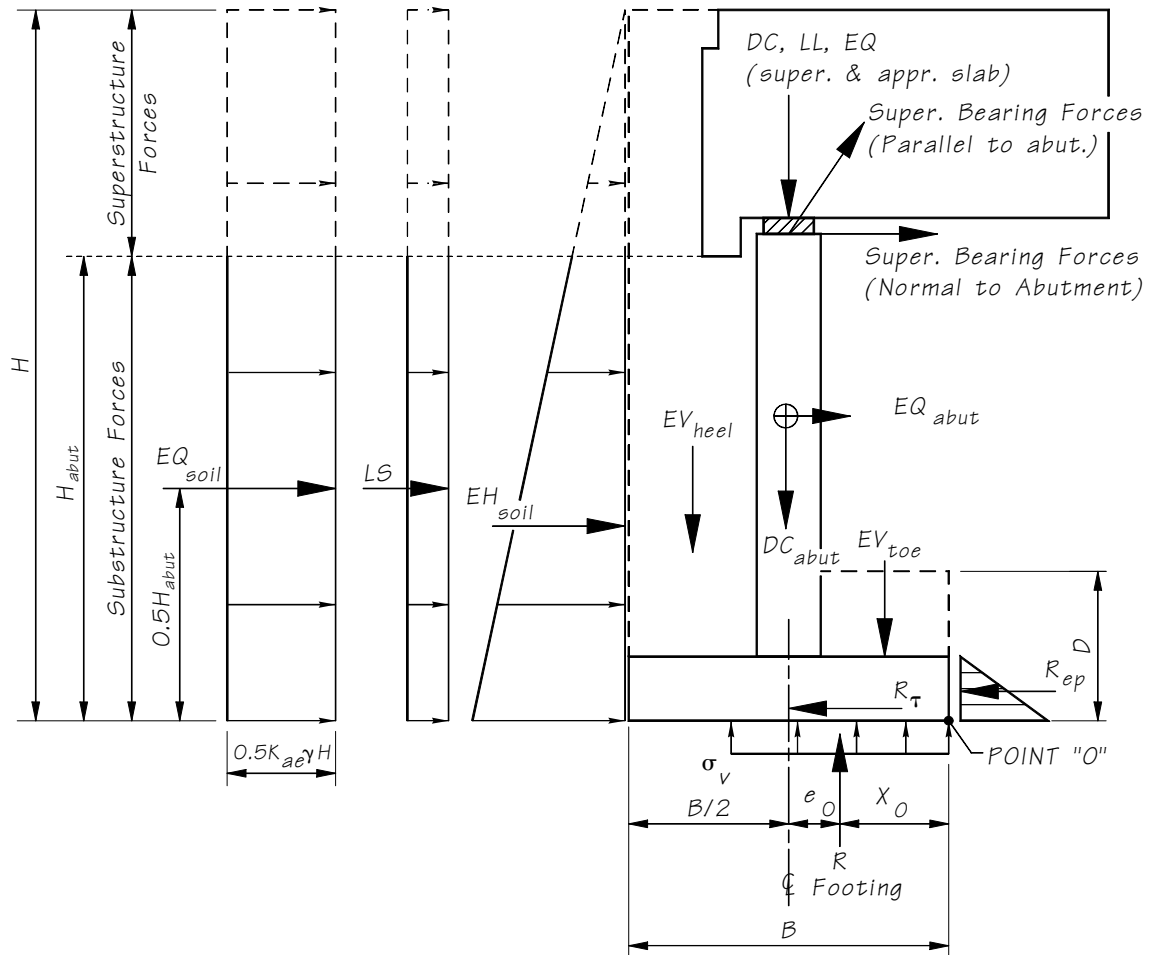
D. Foundation Springs - Extreme Limit State

When a structural evaluation of soil response is required for a bridge analysis, the Geotechnical Branch will determine foundation soil/rock shear modulus and Poissons ratio (G and μ). These values will typically be determined for shear strain levels of 2% to 0.2%, which are typical strain levels for large magnitude earthquakes.

7.7.4 Spread Footing Structural Design

The following BDM Section is oriented towards abutment spread footing design. Spread footing designs for intermediate piers or other applications use the same concepts with the appropriate structural analysis. Structural designers should complete all design checks before consulting with the geotechnical engineer about any design problem. There may be several problem criteria that should be addressed in the solution.

A. Abutment Spread Footing Force Diagram

Figures 7.7.4-1 and 7.7.4-2 diagram the forces that act on abutment footings. Each limit state design check will require calculation of a reaction (R) and the location (X_o) or eccentricity (e_o). The ultimate soil passive resistance (Q_{ep}) at the toe is determined by the geotechnical engineer and is project specific.

Cantilever (End Diaphragm) Abutment Force Diagram
Figure 7.7.4-1

DC, LL (Appr. Slab)

DC, LL, EQ
(superstructure)

Superstructure Forces
(Parallel to abut.)

Superstructure Forces
(Normal to Abutment)

EV_{heel}

Substructure Forces

H

EQ_{soil}

LS

EH_{soil}

$0.5H$

EQ_{abut}

DC_{abut}

EV_{toe}

D

R_T

R_{ep}

σ_v

POINT "O"

$0.5K_{ae}\gamma H$

$B/2$ e_O X_O

R

℄ Footing

B

ALL SOIL PRESSURE RESULTANTS
SHALL BE APPLIED AT THE CENTROIDS
OF THE DIAGRAMS OF PRESSURE
ACTING ON THE ABUTMENT.
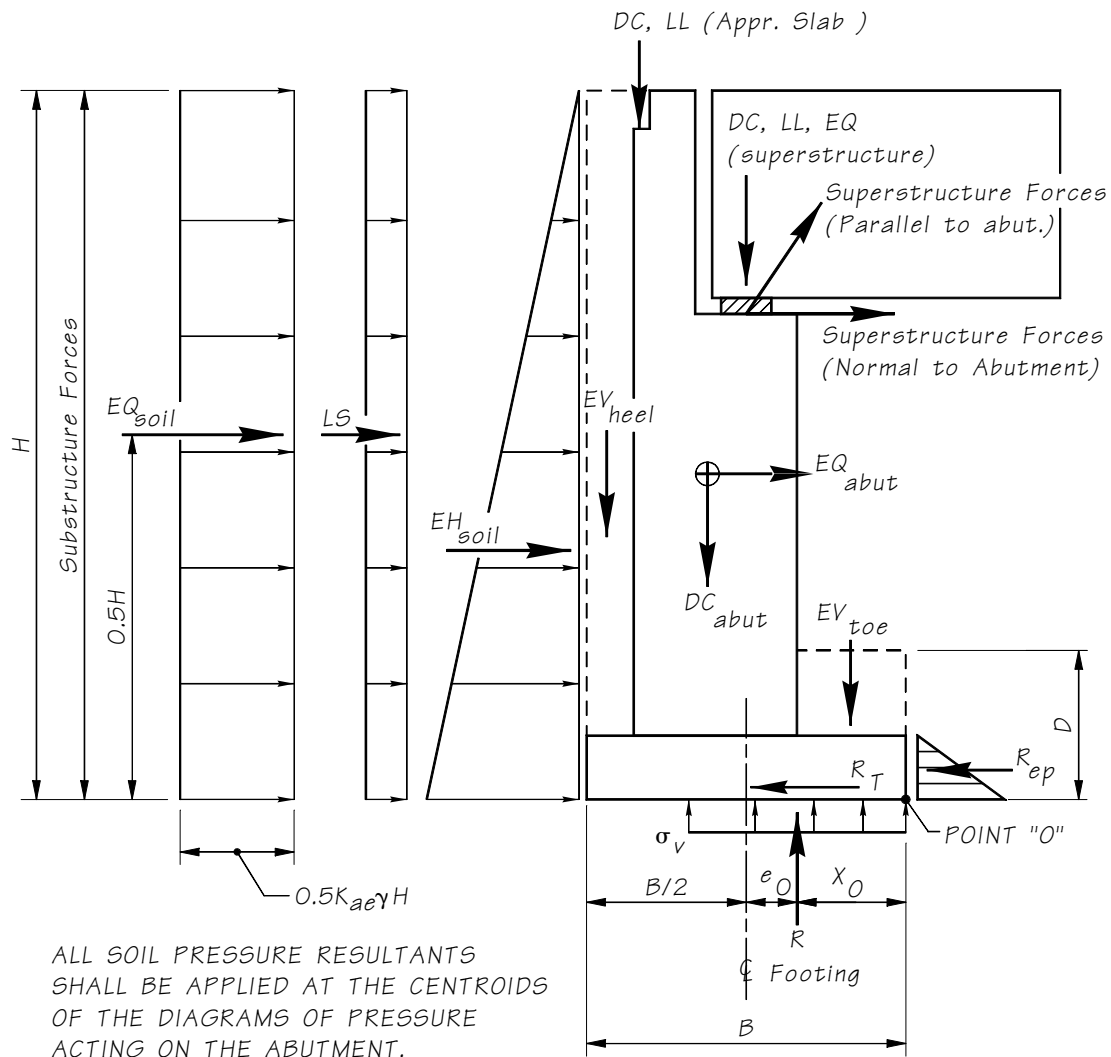
L-Abutment Force Diagram
Figure 7.7.4-2

B   Bearing Stress

For geotechnical and structural footings design, the bearing stress calculation assumes a uniform bearing pressure distribution. For footing designs on rock, the bearing stress is based on a triangular or trapezoidal bearing pressure distribution. The procedure to calculate bearing stress is summarized in the following outline. See Abutment Spread Footing Force Diagrams for typical loads and eccentricity.

Step 1:   Calculate the Resultant force (R_{str}), location (Xo_{str}) and eccentricity for Strength (e_{str}).

Xo_{str} = (factored moments about the footing base)/(factored vertical loads)

Step 2A: For Footings on Soil:

Calculate the maximum soil stress (σ_{str}) based on a uniform pressure distribution. Note that this calculation method applies in both directions for biaxially loaded footings. See AASHTO 10.6.3.1.5 for guidance on biaxial loading. The maximum footing pressure on soil with a uniform distribution is:

$\sigma_{str} = R/B' = R/2Xo = R/(B-2e)$, where B' is the effective footing width.

Step 2B: For Footings on Rock:

If the reaction is outside the middle ⅓ of the base, use a triangular distribution.

σ_{str} max = 2R/3 Xo, where "R" is the factored limit state Reaction.

If the reaction is within the middle ⅓ of the base, use a trapezoidal distribution.

σ_{str} max = R/B $(1+ 6\ e/B^2)$

In addition, WSDOT limits the maximum stress (P/A) applied to rock due to vertical loads only. This is because the rock stiffness approaches infinity relative to the footing concrete. The maximum width of uniform stress is limited to C+2D as shown in Figure 7.7.4-3.

Step 3:   Compare the factored bearing stress (σ_{str}) to the factored bearing capacity ($\phi b_c q_n$) of the soil or rock. The factored bearing stress must be less than or equal to the factored bearing capacity.

$\sigma_{str} \leq \phi b_c q_n$

Step 4:   Repeat steps 1 thru 3 for the Extreme Event limit state. Calculate Xo_{ext}, e_{ext}, and σ_{ext} using Extreme factors and compare the factored stress to the factored bearing ($\phi_b q_n$).
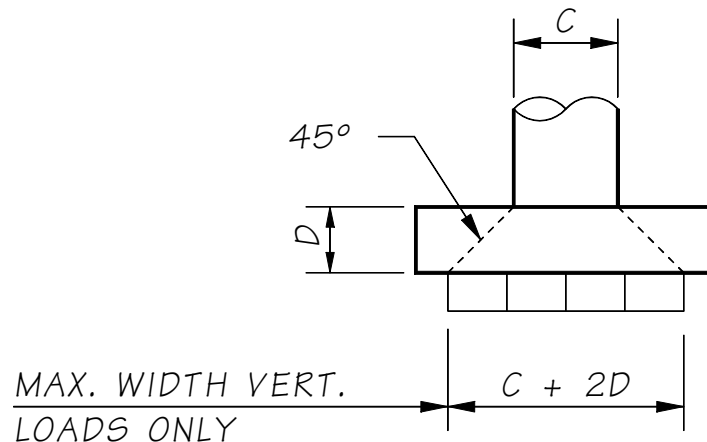
Footings on Rock
Figure 7.7.4-3

C. Failure By Sliding

The factored sliding resistance (Q_R) is comprised of a frictional component ($\varphi_\tau Q_\tau$) and the Geotechnical Branch may allow a passive earth pressure component ($\phi_{ep} Q_{ep}$). The designer shall calculate Q_R based on the soil properties specified in the Geotechnical Report. The frictional component acts along the base of the footing, and the passive component acts on the vertical face of a buried footing element. The factored sliding resistance should be greater than or equal to the factored horizontal applied loads.

$Q_R = \phi_\tau Q_\tau + \phi_{ep} Q_{ep}$

The Strength Limit State ϕ_τ and ϕ_{ep} are provided in the Geotechnical Report or AASHTO 10.5.5-1. The Extreme Event Limit State Q_τ and ϕ_{ep} are generally equal to 1.0.

Q_τ = (R) tan δ
δ = friction angle between the footing base and the soil
δ = tan ϕ for cast-in-place concrete against soil
δ = (0.8)tan ϕ for precast concrete
R = Minimum Strength and Extreme factors are used to calculate R
ϕ = angle of internal friction for soil

D. Overturning Stability

Calculate the locations of the overturning reaction (R) for strength and extreme limit states. Minimum load factors are applied to forces and moments resisting overturning. Maximum load factors are applied to forces and moments causing overturning. Note that for footings subjected to biaxial loading, the following eccentricity requirements apply in both directions.

See AASHTO LRFD Articles 11.6.3.3 (Strength Limit State) and 11.6.5 (Extreme Event Limit State) for the appropriate requirements for the location of the overturning reaction (R)..

E. Footing Settlement

The service limit state bearing capacity (q_{ser}) will be a settlement-limited value, typically 1 inch.

Bearing Stress = $\sigma_{ser} < \phi q_{ser}$ = Factored nominal bearing

Where, q_{ser} is the unfactored service limit state bearing capacity and ϕ is the service resistance factor. In general, the resistance factor (ϕ) shall be equal to 1.0.

For immediate settlement (not time dependent), both permanent dead load and live load should be considered for sizing footings for the service limit state. For long-term settlement (on clays), only the permanent dead loads should be considered.

If the structural analysis yields a bearing stress (σ_{ser}) greater than the bearing capacity, then the footing must be re-evaluated. The first step would be to increase the footing size to meet bearing capacity. If this leads to a solution, recheck layout criteria and inform the geotechnical engineer the footing size has increased. If the footing size cannot be increased, consult the geotechnical engineer for other solutions.

F. Concrete Design

Footing design shall be in accordance with AASHTO Section 5.13.3 for footings and the general concrete design of AASHTO Chapter 5. The following Figure 7.7.4-4 illustrates the modes of failure checked in the footing concrete design.
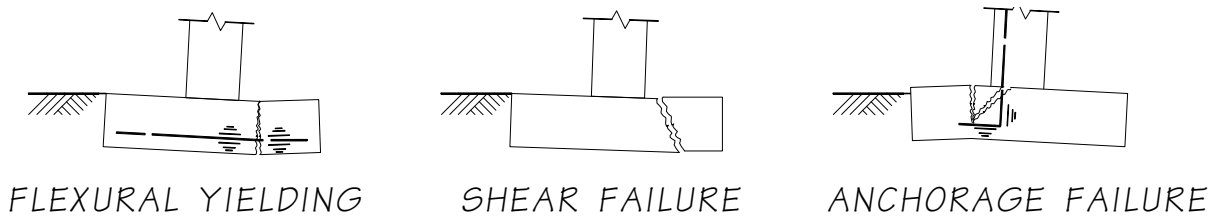
FLEXURAL YIELDING   SHEAR FAILURE   ANCHORAGE FAILURE

Figure 7.7.4-4

1. Footing Thickness and Shear

    The minimum footing thickness shall be 1 foot 6 inches. The minimum plan dimension shall be 4 feet 0 inches. Footing thickness may be governed by the development length of the column dowels, or by concrete shear requirements (with or without reinforcement). If concrete shear governs the thickness, it is the Engineer's judgment, based on economics, as to whether to use a thick footing unreinforced for shear or a thinner footing with shear reinforcement. Generally, shear reinforcement should be avoided but not at excessive cost in concrete, excavation, and shoring requirements. Where stirrups are required, place the first stirrup at d/2 from the face of the column or pedestal. For large footings, consider discontinuing the stirrups at the point where vu = vc.

2. Footing Force Distribution

    The maximum shear stress in the footing concrete shall be determined based on a triangular or trapezoidal bearing pressure distribution, see AASHTO 5.13.3.6. This is the same pressure distribution as for footing on rock, see BDM Section 7.7.4B.

3. Vertical Reinforcement (Column or Wall)

    Vertical reinforcement shall be developed into the footing to adequately transfer loads to the footing. Vertical rebar shall be bent 90° and extend to the top of the bottom mat of footing reinforcement. This facilitates placement and minimizes footing thickness. Bars in tension shall be developed using 1.25 Ld. Bars in compression shall develop a length of 1.25 Ld, prior to the bend. Where bars are not fully stressed, lengths may be reduced in proportion, but shall not be less than ¾ Ld.

    The concrete strength used to compute development length of the bar in the footing shall be the strength of the concrete in the footing. The concrete strength to be used to compute the section strength at the interface between footing and a column concrete shall be that of the column concrete. This is allowed because of the confinement effect of the wider footing.

4. Bottom Reinforcement

    Concrete design shall be in accordance with AASHTO. Reinforcement shall not be less than #6 bars at 12 inch centers to account for uneven soil conditions and shrinkage stresses.

5. Top Reinforcement

    Top reinforcement shall be used in any case where tension forces in the top of the footing are developed. Where columns and bearing walls are connected to the superstructure, sufficient reinforcement shall be provided in the tops of footings to carry the weight of the footing and overburden assuming zero pressure under the footing. This is the uplift earthquake condition described under "Superstructure Loads." This assumes that the strength of the connection to the superstructure will carry such load. Where the connection to the superstructure will not support the weight of the substructure and overburden, the strength of the connection may be used as the limiting value for determining top reinforcement. For these conditions, the AASHTO requirement for minimum percentage of reinforcement will be waived. Regardless of whether or not the columns and bearing walls are connected to the superstructure, a mat of reinforcement shall normally be provided at the tops of footings. On short stub abutment walls (4 feet from girder seat to top of footing), these bars may be omitted. In this case, any tension at the top of the footing, due to the weight of the small overburden, must be taken by the concrete in tension.

    Top reinforcement for column or bearing wall footings designed for two-way action shall not be less than #6 bars at 12 inch centers, in each direction while top reinforcement for bearing wall footings designed for one-way action shall not be less than #5 bars at 12 inch centers in each direction.

7.7.5 Footing Concrete Design on Pile Supports

The minimum footing thickness shall be 2 feet 0 inches. The minimum plan dimension shall be 4 feet 0 inches. Footing thickness may be governed by the development length of the column dowels, or by concrete shear requirements. The use of strut and tie modeling is recommended for the design of all pile caps and pile footings. Figure 7.7.5-1 identifies the modes of failure that should be investigated for general pile cap/footing design.
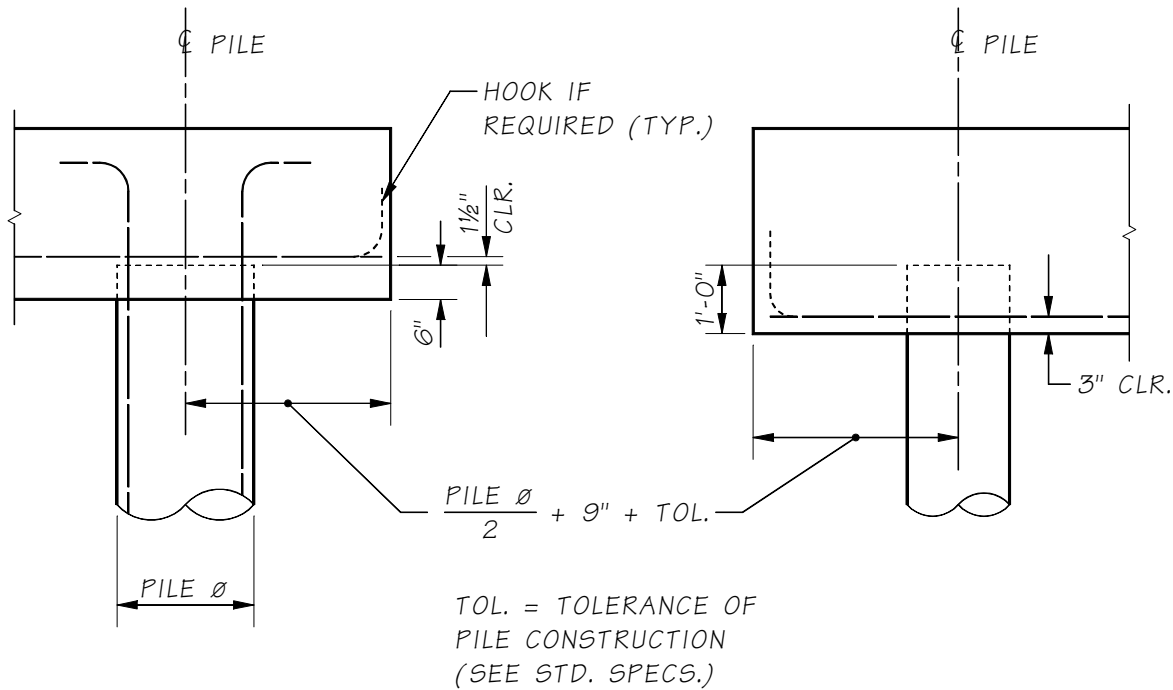
ANCHORAGE FAILURE

CONCRETE SHEAR FAILURE

FLEXURAL YIELDING

PILE PULLOUT
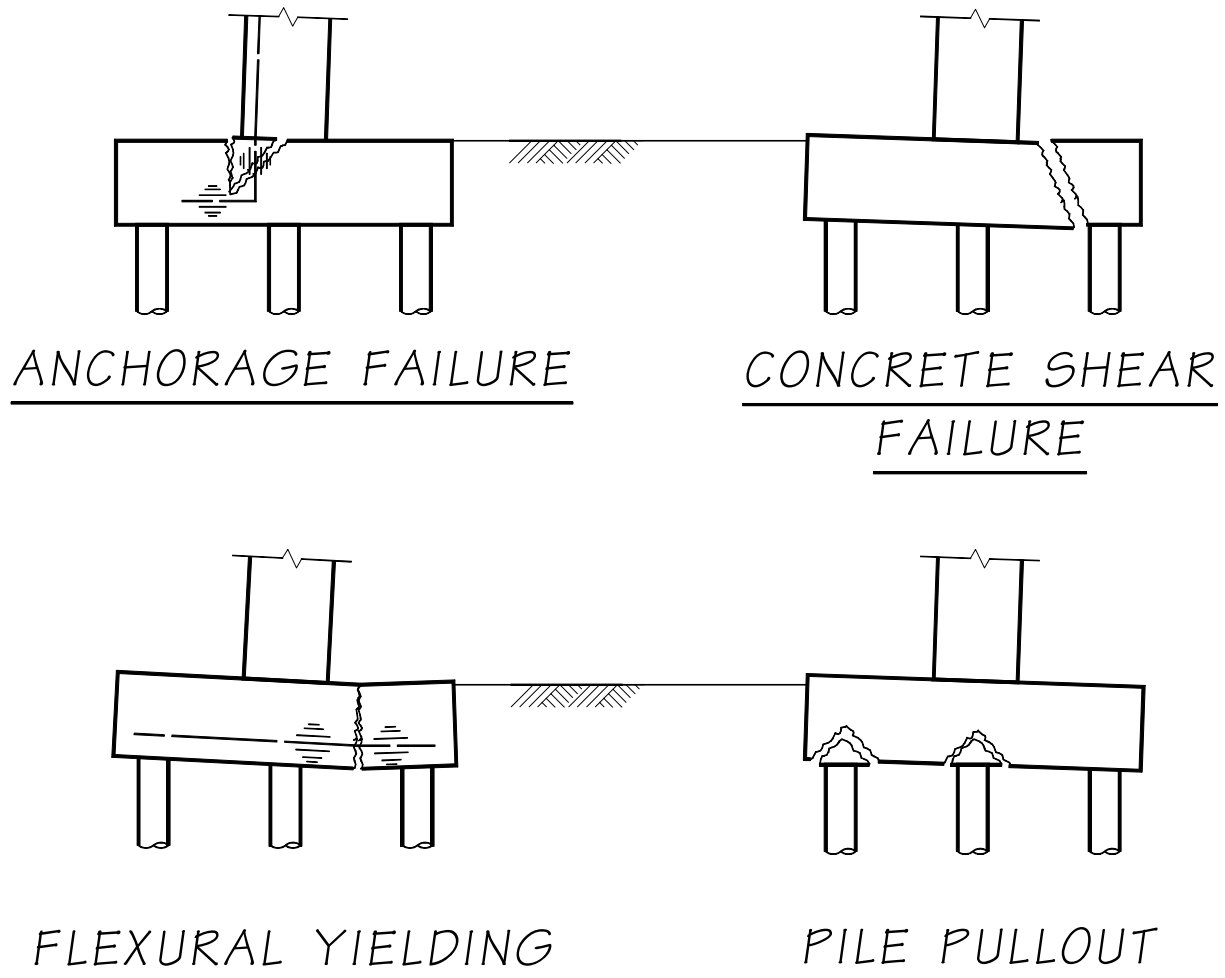
Pile Footing Modes of Failure
Figure 7.7.5-1

A. Pile Embedment, Clearance, and Rebar Mat Location

All piles shall have an embedment in the concrete sufficient to resist moment, shear, and axial loads. Cast-in-place concrete piles with reinforcing extending into footings are embedded a minimum of 6 inches. The clearance for the bottom mat of footing reinforcement shall be 1½" between the reinforcing and the top of the pile for C.I.P. pile footings. See Figure 7.7.5-2 for the minimum pile clearance to the edge of footing.

Pile Embedment and Reinforcing Placement
Figure 7.7.5-2

B. Concrete Design

In determining the proportion of pile load to be used for calculation of shear stress on the footing, any pile with its center 6 inches or more outside the critical section shall be taken as fully acting on that section. Any pile with its center 6 inches or more inside the critical section shall be taken as not acting for that section. For locations in between, the pile load acting shall be proportioned between these two extremes. The critical section shall be taken as the effective shear depth (d_v) as defined in AASHTO LRFD 5.8.2.9. The distance from the column/wall face to the allowable construction centerline of pile (design location plus or minus the tolerance) shall be used to determine the design moment of the footing. The strut and tie design method should be used where appropriate.

7.8   Drilled Shafts

7.8.1 Axial Resistance

The factored axial resistance of the drilled shaft (R) is generally composed of two parts: the nominal end bearing (R_p) and the nominal skin friction (R_s). The general formula is as follows, where φ is the limit state resistance factor.

$$R = \varphi_p \, R_p + \varphi_s \, R_s$$

The total factored shaft loading must be less than the factored axial resistance. R_p and R_s are treated as independent quantities although research has shown that the end bearing and skin friction resistance have some interdependence. R_p and R_s will be stated in the Geotechnical Report for the bridge.

The Bridge Plans shall include the end bearing and skin friction capacity for the service, strength, and extreme event limit states in the General Notes, as shown in

THE NOMINAL SHAFT CAPACITY SHALL BETAKENAS, IN KIPS:

SERVICE-I LIMIT STATE		
PIER NO.	SKIN FRICTION CAPACITY	END BEARING CAPACITY
1	====	====
2	====	====

STRENGTH LIMIT STATE		
PIER NO.	SKIN FRICTION CAPACITY	END BEARING CAPACITY
1	====	====
2	====	====

EXTREME EVENT-I LIMIT STATE		
PIER NO.	SKIN FRICTION CAPACITY	END BEARING CAPACITY
1	====	====
2	====	====

Figure 7.8.1-1

7.8.2   Structural Design and Detailing

The current ADSC/WSDOT Shaft Special Provision should be reviewed as part of the design of drilled shafts. The structural design of drilled shafts is similar to column design. The following guidelines shall be followed:

A.   Drilled shafts shall be designed for the lesser of the plastic forces or elastic seismic forces of the column above in single column/single shaft foundations. This applies to all seismic zones in Washington State.

B.   Concrete Class 4000P shall be specified for the entire length of the shaft, wet or dry conditions of placement.

C.   When shafts are constructed in water, the concrete specified for the casing shoring seal shall be Class 4000W.

D.   The assumed concrete compressive strength shall be 0.85f'c for structural design of shafts. Most shafts in the State are constructed with the wet method using slurries to stabilize caving soils. A reduction in concrete strength is used to account for the unknown shaft concrete quality that results.

E.   The presence of permanent steel casing shall be taken into account in the shaft design (i.e. for stiffness, and etc.), but the structural capacity of permanent steel casing shall not be considered for structural design of drilled shafts.

F.   Cover requirements vary, depending on the drilled shaft diameter. See subsection 3.05.C of the current ADSC/WSDOT Shaft Special Provision for the most current cover requirements.

G.   In general, drilled shaft reinforcing shall be detailed to minimize congestion, facilitate concrete placement by tremie, and maximize consolidation of concrete.

H.   The clear spacing between spirals and hoops shall not be less than 6" or more than 9", with the following exception. The clear spacing between spirals or hoops may be reduced in the splice zone in single column/single shaft connections because shaft concrete may be vibrated in this area, negating the need for larger openings to facilitate good flow of concrete through the reinforcing cage.

I.   The volumetric ratio and spacing requirements of the AASHTO Guide Specifications for LRFD Seismic Bridge Design for confinement need not be met. The top of shafts in typical WSDOT single column/single shaft connections remains elastic under seismic loads due to the larger shaft diameter (as compared to the column). Therefore this requirement does not need to be met.

J.   Shaft transverse reinforcement may be constructed as hoops or spirals. Spiral reinforcement is preferred for shaft transverse reinforcement. However, if #6 spirals at 6" (excluding the exception in 7.8.2H) clear do not satisfy the shear design, hoops may be used. Full welded splices as shown in Figure 7.8.2-1 shall be used.
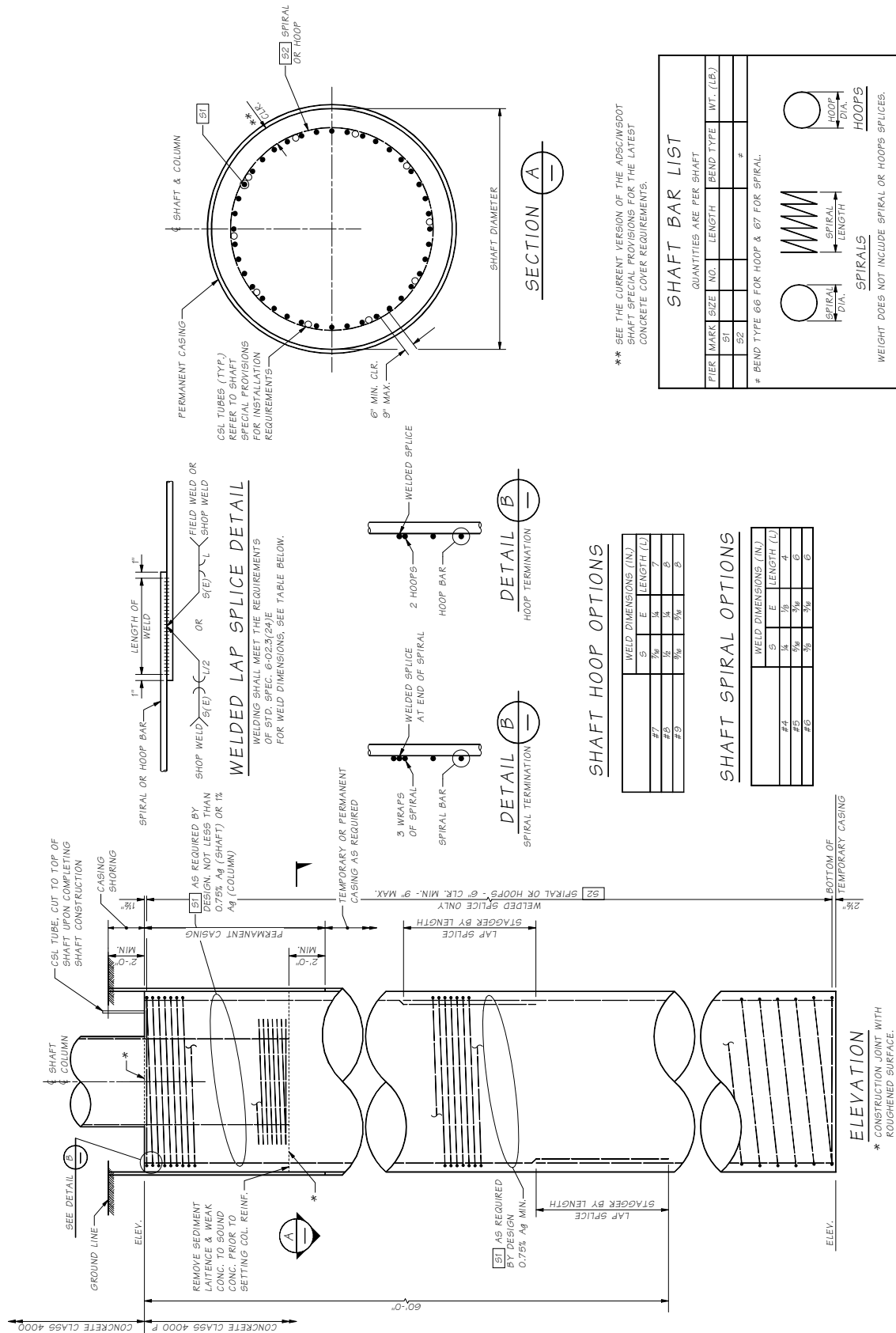
Typical Drilled Shaft Details
Figure 7.8.2-1

K. In single column/single shaft configurations, the spacing of the shaft transverse reinforcement in the splice zone shall meet the requirements of the following equation, which comes from the TRAC Report titled, "NONCONTACT LAP SPLICES IN BRIDGE COLUMN-SHAFT CONNECTIONS":

$$s_{tr} = \frac{2\pi A_{sp} f_{ytr} l_s}{A_l f_{ul}}$$

Where:

S_{tr} = spacing of transverse shaft reinforcement
A_{sp} = Area of shaft spiral or transverse reinforcement
f_{ytr} = yield strength of shaft transverse reinforcement
l_s = standard splice length of the column reinforcement
A_l = Area of longitudinal shaft reinforcement
f_{ul} = ultimate strength of shaft longitudinal reinforcement

L. Longitudinal reinforcement shall be provided for the full length of drilled shafts. The minimum longitudinal reinforcement in the splice zone of single column/single shaft connections shall be the larger of 0.75% A_g of the shaft or 1.0% A_g of the attached column. The minimum longitudinal reinforcement beyond the splice zone shall be 0.75% A_g of the shaft. The minimum longitudinal reinforcement in shafts without single column/single shaft connections shall be 0.75% A_g of the shaft.

M. The clear spacing between longitudinal reinforcement shall not be less than 6" or more than 9". If a shaft design is unable to meet this minimum requirement, a larger diameter shaft shall be considered.

N. Longitudinal reinforcing in drilled shafts should be straight with no hooks to facilitate concrete placement and removal of casing. If hooks are necessary to develop moment at the top of a drilled shaft (in a shaft cap situation) the hooks should be turned toward the center of the shaft while leaving enough opening to allow concrete placement with a tremie.

O. Use of two concentric circular rebar cages shall be avoided.

P. Resistance factors for Strength Limit States shall be per the latest AASHTO LRFD Bridge Design Specifications. Resistance factors for Extreme Event Limit States shall be per the latest AASHTO Guide Specifications for LRFD Seismic Design.

Q. The axial load along the shaft varies due to the side friction. It is considered conservative, however, to design the shaft for the full axial load plus the maximum moment. The entire shaft normally is then reinforced for this axial load and moment.

R. The resistance factor for shear shall conform to the AASHTO LRFD Bridge Design Specifications.

S. Access tubes for Crosshole Sonic Log (CSL) testing shall be provided in all shafts. One tube shall be furnished and installed for each foot of shaft diameter, rounded to the nearest whole number, and shown in the plans. The number of access tubes for shaft diameters specified as "X feet 6 inches" shall be rounded up to the next higher whole number. The access tubes shall be placed around the shaft, inside the spiral or hoop reinforcement and three inches clear of the vertical reinforcement, at a uniform spacing measured along the circle passing through the centers of the access tubes. If the vertical reinforcement is not bundled and each bar is not more than one inch in diameter, the access tubes shall be placed two inches clear of the vertical reinforcement. If these minimums cannot be met due to close spacing of the vertical reinforcement, then access tubes shall be bundled with the vertical reinforcement.

T. Shafts shall be specified in English dimensions and shall be specified in sizes that do not preclude any drilling method. Shafts shall be specified in whole foot increments except as allowed here. The tolerances in the current ADSC/WSDOT Shaft Special Provision accommodate Metric casing sizes for shafts specified as 2', 3', 4', 7', 8', and 9' diameter. See Table 7.8.2-1.

Nominal (Outside) English Casing Diameter (feet)	Nominal English Casing Diameter (inches)	* Maximum Increase in Casing Inside Diameter (inches)	Maximum English Casing Diameter (inches)	Nominal (Outside) Metric Casing Diameter (meters)	Nominal Metric Casing Diameter (feet)	Nominal Metric Casing Diameter (inches)
10.0	120	6	126			
9.5	114	6	120	3.00	9.84	118.11
9.0	108	6	114	2.80	9.19	110.23
8.0	96	6	102	2.50	8.20	98.42
7.0	84	6	90	2.20	7.22	86.61
6.5	78	6	84	2.00	6.56	78.74
6.0	72	6	78			
5.5	66	6	72			
5.0	60	12	72			
4.5	54	12	66	1.50	4.92	59.05
4.0	48	12	60	1.50	4.92	59.05
3.0	36	12	48	1.00	3.28	39.37
3.0	36	12	48	0.915	3.00	36.02
2.0	24	12	36	0.70	2.30	27.56

  * Check the current ADSC/WSDOT Shaft Special Provision

Table 7.8.2-1

As seen in Table 7.8.2-1. Metric casings are not readily available to accommodate shafts specified as 5'-0", 6'-0", and 10'-0" diameter. For such cases, the preferred approach is to design and specify the shafts as 4'-6", 6'-6", and 9'-6" diameter shafts. The construction tolerances in the current Shaft Special Provision would then also allow contractors to either up-size to 5'-0", 7'-0", and 10'-0" diameter shafts, respectively, or to furnish the appropriate metric casing. Alternatively, two shaft designs may be shown in the plans if 5'-0", 6'-0", or 10'-0" diameter shafts are desired. One of the designs shall accommodate a Metric casing, but shall be specified in English to the nearest half-foot diameter. Metric casing sizes are typically available in the sizes shown in Table 7.8.2-1.

U. Shafts supporting a single column shall be sized to allow for construction tolerances, as illustrated in Figure 7.8.2-2.
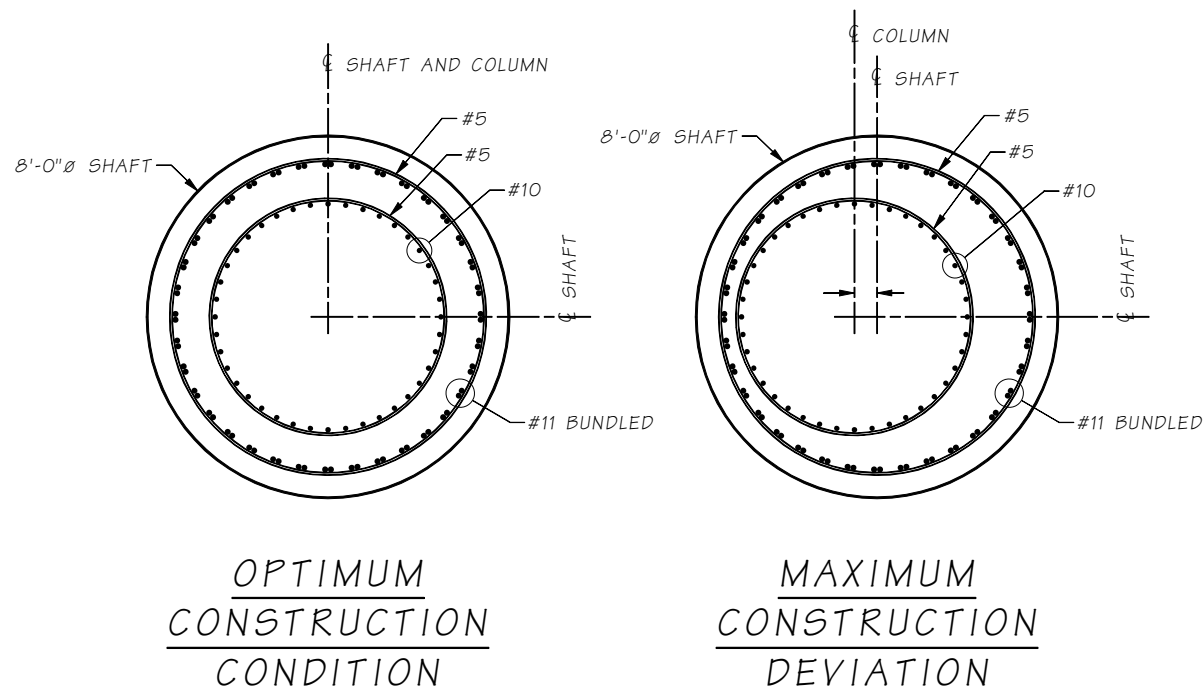

Figure 7.8.2-2

The shaft diameter shall be based on the maximum column diameter allowed by the following equation,

Maximum Column Diameter = Shaft Diameter– 2*(Shaft Concrete Cover) – 2*(Shaft Horizontal Construction Tolerance) – 2*(Shaft Cage Thickness)

The shaft horizontal construction tolerance and shaft concrete cover shall be per the current ADSC/WSDOT Shaft Special Provision.

If the column diameter used in design is larger than the maximum allowed for a given shaft size, as defined by the equation above, a larger shaft diameter shall be used.

The shaft diameter specified here should not be confused with the desirable casing shoring diameter discussed below.

V. Casing shoring shall be provided for all shafts below grade or waterline. However, casing shoring requirements are different for shafts in shallow excavations and deep excavations. Shafts in deep excavations require a larger diameter casing shoring to allow access to the top of the shaft for column form placement and removal. The top of shafts in shallow excavations (approximately 4 feet or less) can be accessed from the ground line above, by reaching in or by "glory-holing", and therefore do not require larger diameter casing shoring. See Figure 7.8.2-3.
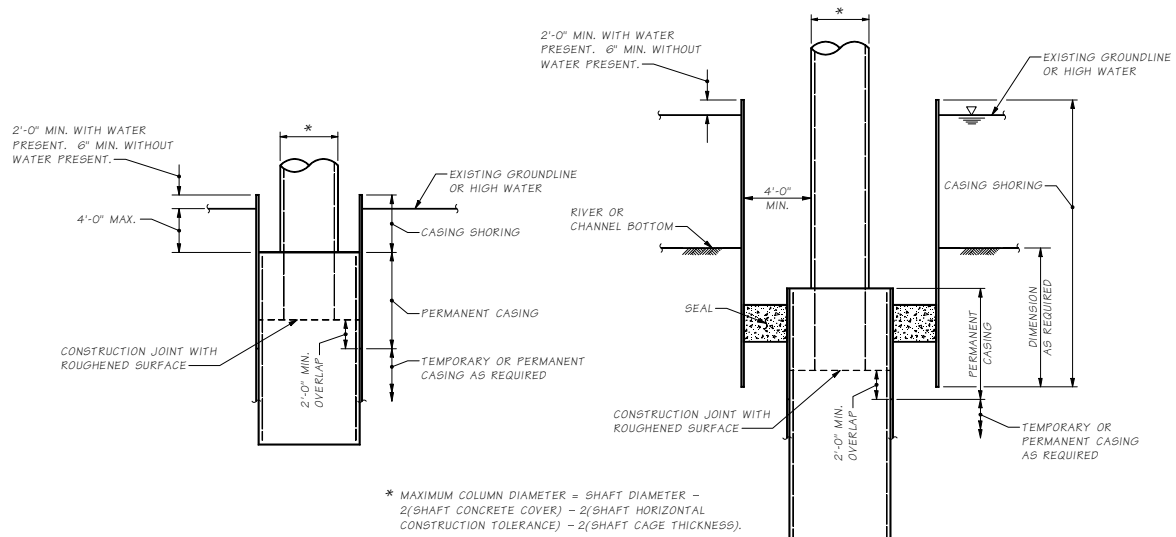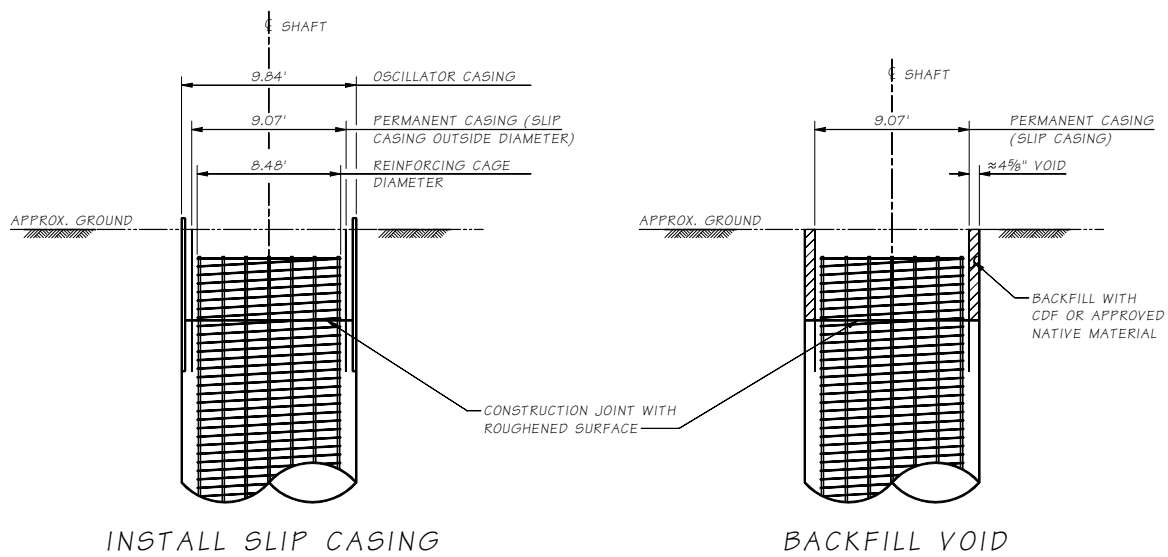
Figure 7.8.2-3

W. Changes in shaft diameters due to construction tolerances allowed in the ADSC/WSDOT Shaft Special Provision shall not result in a reinforcing steel cage diameter different from the diameter shown in the plans (plan shaft diameter minus concrete cover). For example, Metric casing diameters used in lieu of English casing diameters shall only result in an increase in concrete cover, except as noted below for single column/single shaft connections requiring slip casings.

X. Rotator and Oscillator drilling methods typically use a slip casing for permanent casing in single column/single shaft connections, as shown in Figure 7.8.2-4.

9'-6"Ø SHAFT CONSTRUCTED
WITH THE OSCILLATOR METHOD

Figure 7.8.2-4

The use of the slip casing typically requires a modification to the reinforcing cage diameter. This should be considered during the structural design of the shaft. The slip casing also results in less concrete cover than the area of the shaft below the slip casing. See Table 7.8.2-2 for expected reinforcing cage diameters and clear cover. Shafts shall be designed such that the reduced concrete cover is acceptable in this area because the casing is permanent. A minimum of 2.50" of concrete cover is achievable in this area and shall be kept as a minimum requirement. The reduction in strength (compared to the area below the slip casing) associated with the reduced shaft diameter that results from the slip casing is bounded within the shaft analysis and design methods prescribed here and elsewhere. Therefore the reduction in strength in this area can be ignored.

Nominal (Outside) Metric Casing Diameter (meters)	Nominal (Outside) Metric Casing Diameter (feet)	¹Nominal (Outside) Metric Cage Diameter (inches)	Nominal (Outside) Metric Cage Diameter (feet)	²Inside Diameter of Metric Casing (inches)	³Nominal (Outside) Metric Slip Casing Diameter (inches)	Nominal (Outside) Metric Slip Casing Diameter (feet)	Nominal (Outside) Metric Slip Casing Diameter (meters)	Cage Clearance Below Slip Casing (inches)	⁴Cage Clearance at Slip Casing (inches)
3.00	9.84	101.81	8.48	111.81	108.81	9.07	2.76	8.15	2.50
2.80	9.19	95.51	7.96	105.51	102.51	8.54	2.60	7.36	2.50
2.50	8.20	83.70	6.98	93.70	90.70	7.56	2.30	7.36	2.50
2.20	7.22	71.89	5.99	81.89	78.89	6.57	2.00	7.36	2.50
2.00	6.56	64.02	5.34	74.02	71.02	5.92	1.80	7.36	2.50
1.50	4.92	45.12	3.76	55.12	52.12	4.34	1.32	6.97	2.50
1.50	4.92	45.12	3.76	55.12	52.12	4.34	1.32	6.97	2.50
1.00	3.28	26.22	2.19	36.22	34.22	2.85	0.87	6.57	3.00
0.92	3.00	22.87	1.91	32.87	30.87	2.57	0.78	6.58	3.00

Table 7.8.2-2

Notes:

1 Provided by Malcolm Drilling. Assumes minimum of 5 inches clearance to inside of oscillator casing on 5 foot and larger and uses 3 inches of clearance on 4 foot and smaller.

2 Provided by Malcolm Drilling.

3 Provided by Malcolm Drilling. Slip Casing is 3" smaller than ID of temporary casing from 1.2M to 3M. 1M on down is 2" smaller in diameter.

4 Slip casing is typically ⅜" to ½" thick (Provided by Malcolm Drilling). Cage clearance assumes ½" thick casing.

Y. Reinforcing bar centralizers shall be detailed in the plans as shown in Figure 7.8.2-5. The centralizers shall be detailed as ½" less than the concrete cover required in the current ADSC/WSDOT Shaft Special Provision.
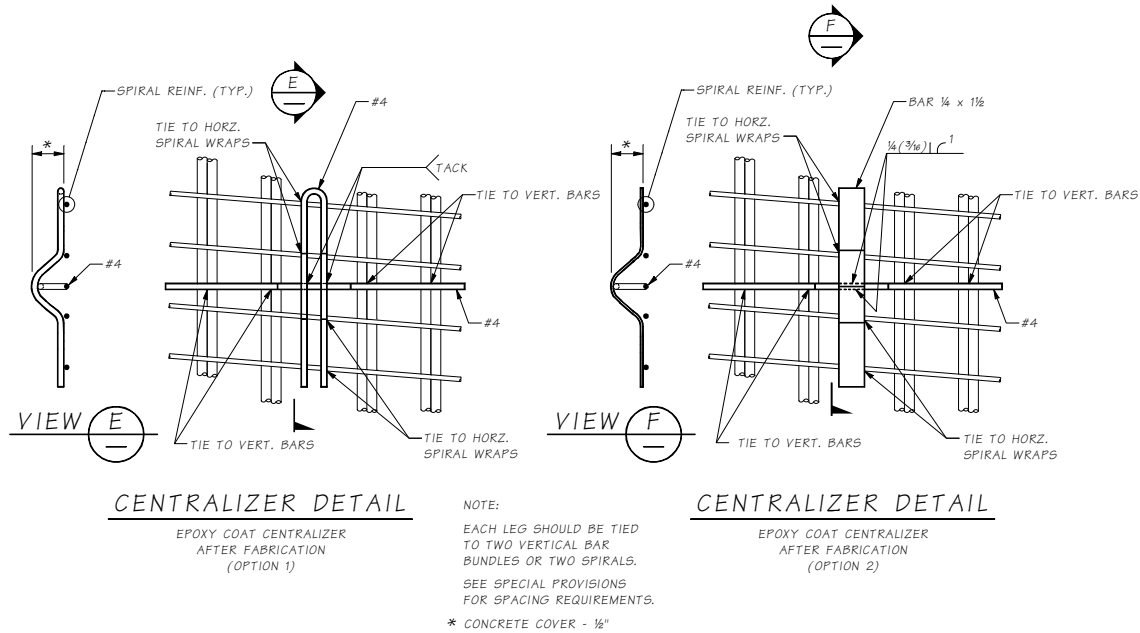

Figure 7.8.2-5

7.9 Piles and Piling

7.9.1 Pile Types

This section of the BDM describes the piling used by the Bridge and Structures Office and their applications. In general, piles should not be used where spread footings can be used. However, where heavy scour conditions may occur, pile foundations should be considered in lieu of spread footings. Also, where large amounts of excavation may be necessary to place a spread footing, pile support may be more economical.

A. Cast-in-Place Concrete Piles

Cast-in-Place (CIP) concrete piles utilize driven steel pipe casings, which are then filled with reinforcing steel and concrete. The bottom of the casing is typically capped with a suitable flat plate for driving. However, the Geotechnical Branch may specify special tips when difficult driving is expected.

The Geotechnical Branch will determine the minimum wall thickness of the steel pipe casings based on driving conditions. However, the Standard Specifications require the contractor to provide a wall thickness that will prevent damage during driving.

B. Precast, Prestressed Concrete Piles

Precast, prestressed concrete piles are octagonal, or square in cross-section and are prestressed to allow longer handling lengths and resist driving stresses. Standard Plans are available for these types of piles.

C. Steel H Piles

Steel piles have been used where there are hard layers that must be penetrated in order to reach an adequate point bearing stratum. Steel stress is generally limited to 9.0 ksi (working stress) on the tip. H piling can act efficiently as friction piling due to its large surface area. Do not use steel H piling where the soil consists of only moderately dense material. In such conditions, it may be difficult to develop the friction capacity of the H piles and excessive pile length may result.

D. Timber Piles

Timber piles may be untreated or treated. Untreated piles are used only for temporary applications or where the entire pile will be permanently below the water line. Where composite piles are used, the splice must be located below the permanent water table. If doubt exists as to the location of the permanent water table, treated timber piles shall be used.

Where dense material exists, consideration should be given to allowing jetting (with loss of uplift capacity), use of shoes, or use of other pile types.

E. Steel Sheet Piles

Steel sheet piles are typically used for cofferdams and shoring and cribbing, but are usually not made a part of permanent construction.

Cast-in-place (CIP) concrete piles consisting of steel casing filled with reinforcing steel and concrete are the preferred type of piling for WSDOT's permanent bridges. Other pile types such as precast, prestressed concrete piles, steel H piles, timber piles, auger cast piles, and steel pipe piles shall not be used for WSDOT permanent bridge structures. These types of piles may be used for temporary bridges and other non-bridge applications subject to approval by the State Geotechnical Engineer and the State Bridge Design Engineer.

Micropiles shall not be used for new bridge foundations. This type of pile may be used for foundation strengthening of existing bridges, temporary bridges and other non-bridge applications subject to approval by the State Geotechnical Engineer and the State Bridge Design Engineer.

Battered piles shall not be used for bridge foundations to resist lateral loads.

The above limitations apply to all WSDOT bridges including mega projects and design-build contracts.

The above policy on pile types is the outcome of lengthy discussions and meetings between the bridge design, construction and geotechnical engineers. These limitations are to ensure improved durability, design and construction for WSDOT pile foundations.

In seismic applications there is a need for bi-directional demands. Steel H piles have proven to have little bending capacity for the purposes of resisting seismic load while circular CIP piles provide consistent capacities in all directions. Also, CIP pile casing is generally available in a full range of casing diameters. CIP piles are easily inspected after driving to ensure the quality of the finished pile prior to placing reinforcing steel and concrete. All bending strength is supplied by elements other than the casing in accordance with BDM policy.

Precast, prestressed concrete piles, and timber piles are difficult to splice and for establishing moment connections into the pile cap.

Micropiles have little bending capacity for the purposes of resisting lateral loads in seismic applications.

7.9.2   *Single Pile Axial Resistance*

The Geotechnical Report will provide the nominal axial resistance (R_n) and resistance factor (ϕ) for pile design. The factored pile load ($P_{u\,pile}$) must be less than the factored resistance, ϕR_n, specified in the Geotechnical Report.

Pile axial loading ($P_{u\,pile}$) due to loads applied to a pile cap are determined as follows:

$$(P_{U\,pile}) = (P_{U\,pile\,group})/N + M_{U\,group}\,C/I_{group} + \gamma DD$$

where,

$M_{U\,group}$	=	Factored moment applied to the pile group. This includes eccentric LL, DC, centrifugal force (CE), etc. Generally, the dynamic load allowance (IM) does not apply.
C	=	Distance from the centroid of the pile group to the center of the pile under consideration.
I_{group}	=	Moment of inertia of the pile group
N	=	Number of piles in the pile group
$P_{U\,pile\,group}$	=	Factored axial load to the pile group
DD	=	Downdrag force specified in the Geotechnical Report
γ	=	Load factor specified in the Geotechnical Report

Pile selfweight is typically neglected. As shown above, downdrag forces are treated as load to the pile when designing for axial capacity. However, it should not be included in the structural analysis of the bridge.

7.9.3   Block Failure

For the strength and extreme event limit states, if the soil is characterized as cohesive, the pile group capacity should also be checked for the potential for a "block" failure, as described in AASHTO LRFD Article 10.7.3.9. This check, Step 9 in Figure 7.9.2-1, requires interaction between the designer and the geotechnical engineer. The check is performed by the geotechnical engineer based on loads provided by the designer. If a block failure appears likely, the pile group size should be increased so that a block failure is prevented.

7.9.4   Pile Uplift

Piles may be designed for uplift if specified in the Geotechnical Report. In general, pile construction methods that require preboring, jetting, or spudding must will reduce uplift capacity.

7.9.5   Pile Spacing

Pile spacing determination is typically determined collaboratively with the geotechnical engineer. The Geotechnical Design Manual (GDM) specifies a minimum center-to-center spacing of 30 inches or 2.5 pile diameters. However, center-to-center spacings of less than 2.5 pile diameters may be considered on a case-by-case basis.

7.9.6   Structural Design and Detailing of CIP Concrete Piles

The structural design and detailing of CIP Concrete piles is similar to column design with the following guidelines:

A. Class 4000P Concrete shall be specified for CIP concrete piles. The top 10 feet of concrete in the pile is to be vibrated. Use 1.0 f'c for the structural design.

B. For structural design, the reinforcement alone shall be designed to resist the total moment throughout the length of pile without considering strength of the steel casing. The minimum reinforcement shall be 0.5% Ag for Seismic Zones 1 & 2, and 0.75% A_g for Seismic Zones 3 & 4 as described in AASHTO LRFD Article 5.13.4.6. Minimum clearance between longitudinal bars shall meet the requirements in BDM Ch. 5, Figure 5-A-2.

C. If the pile to footing/cap connection is not a plastic hinge zone longitudinal reinforcement need only extend above the pile into the footing/cap a distance equal to 1.0 l_d (tension). If the pile to footing/cap connection is a plastic hinge zone longitudinal reinforcement shall extend above the pile into the footing/cap a distance equal to 1.25 l_d.

D. Since the diameter of the concrete portion of the pile is dependent on the steel casing thickness, the as-built diameter will not be known during design (since the casing thickness is determined by the contractor). As such, a casing thickness must be assumed for design. The structural engineer should work closely with the geotechnical engineer to determine a suitable casing thickness to assume based on expected driving conditions. A pile drivability analysis may be required for this. Otherwise, the following can typically be assumed:
    - ¼" inch for piles less than 14" in diameter
    - ⅜" inches for piles 14" to 18" in diameter
    - ½" inch for larger piles.

E.   Steel casing for cast-in-place piling should be designated by nominal diameter, not inside diameter for 24 inch and smaller pile casings. Standard Specification Section 9-10.5 requires steel casings to meet ASTM A252, which is purchased by nominal diameter (outside diameter) and wall thickness. A pipe thickness should not be stated in the plans. As stated previously, the Standard Specifications require the contractor to determine the pile casing thickness required for driving.

F.   Transverse spiral reinforcement shall be designed to resist the maximum shear in the pile. Avoid a spiral pitch of less than 3". The minimum spiral shall be a #4 bar at 9" pitch. If the pile to footing/cap connection is not a plastic hinge zone the volumetric requirements of AASHTO LRFD Article 5.13.4.6 need not be met.

G.   Resistance factors for Strength Limit States shall be per the latest AASHTO LRFD Bridge Design Specifications. Resistance factors for Extreme Event Limit States shall be per the latest AASHTO Guide Specifications for LRFD Seismic Design.

H.   Piles are typically assumed to be continuously supported. Normally, the soil surrounding a foundation element provides sufficient bracing against a buckling failure. Piles that are driven through very weak soils should be designed for reduced lateral support, using information from the Geotechnical Division as appropriate. AASHTO LRFD Article 10.7.4.2 may be used to estimate the column length for buckling. Piles driven through firm material normally can be considered fully supported for column action (buckling not critical) below the ground.

I.   The axial load along the pile varies due to side friction. It is considered conservative, however, to design the pile for the full axial load plus the maximum moment. The entire pile is then typically reinforced for this axial load and moment.

J.   In all cases of uplift, the connection between the pile and the footing must be carefully designed and detailed. The bond between the pile and the seal may be considered as contributing to the uplift resistance. This bond value shall be limited to 10 psi. The pile must be adequate to carry tension throughout its length. For example, a timber pile with a splice sleeve could not be used.

7.9.7   Pile Splices

Pile splices shall be avoided where possible. If splices may be required in timber piling, a splice shall be detailed on the plans. Splices between treated and untreated timber shall always be located below the permanent water line. Concrete pile splices shall have the same strength as unspliced piles.

7.9.8   Pile Lateral Design

The strength limit state for lateral resistance is only structural, though the determination of pile fixity is the result of soil-structure interaction. A failure of the soil does not occur; the soil will continue to displace at constant or slightly increasing resistance. Failure occurs when the pile reaches the structural limit state and this limit state is reached, in the general case, when the nominal combined bending, shear, and axial resistance is reached.

Piles resist horizontal forces by a combination of internal strength and the passive pressure resistance of the surrounding soil. The capacity of the pile to carry horizontal loads should be investigated using a soil/structural analysis. For more information on modeling individual piles or pile groups, see BDM Section 7.2, Foundation Modeling.

7.9.9   Battered Piles

As stated previously, battered piles shall not be used to resist lateral loads for new bridge foundations. Where battered piles are used, the maximum batter shall be 4½:12. Piles with batters in excess of this become very difficult to drive and the bearing values become difficult to predict. Ensure that battered piling do not intersect piling from adjacent footings within the maximum length of the piles.

7.9.10   Pile Tip Elevations and Quantities

Pile length quantities provided to PS&E are based on the estimated tip elevation given in the Geotechnical Report or the depth required for design whichever is greater. If the estimated tip elevation given in the Geotechnical Report is greater than the design tip elevation, overdriving the pile will be required. The Geotechnical Engineer should be contacted to evaluate driving conditions. Bridge Special Provision BSP050311D5.FB6 is required in the Special Provisions to alert the contractor of the additional effort needed to drive these piles.

Minimum pile tip elevations provided in the Geotechnical Report may need to be adjusted to lower elevations depending on the results of the lateral, axial, and uplift analysis. This would become the minimum pile tip elevation requirement for the contract specifications. If adjustment in the minimum tip elevations is necessary, or if the pile diameter needed is different than what was assumed for the Geotechnical Report, the Geotechnical Branch MUST be informed so that pile drivability can be re-evaluated.

Note that lateral loading and uplift requirements may influence (possibly increase) the number of piles required in the group if the capacity available at a reasonable minimum tip elevation is not adequate. This will depend on the soil conditions and the loading requirements. For example, if the upper soil is very soft or will liquefy, making the minimum tip elevation deeper is unlikely to improve the lateral response of the piles enough to be adequate. Adding more piles to the group or using a larger pile diameter to increase the pile stiffness may be the only solution.

7.9.11   Plan Pile Resistance

The Bridge Plan General Notes shall list the Ultimate Bearing Capacity (Nominal Driving Resistance, R_{ndr}) in tons. This information is used by the contractor to determine the pile casing thickness and size the hammer to drive the piles. The resistance for several piers may be presented in a table as shown in Figure 7.9.12-1. If overdriving the piles is required to reach the minimum tip elevation, the estimated amount of overdriving (tons) shall be specified in the Special Provisions with BSP050311D5.FB6.

THE PILES SHALL BE DRIVEN TO AN ULTIMATE BEARING CAPACITY AS FOLLOWS:	
PIER NO.	ULTIMATE BEARING CAPACITY (TONS)
1	====
4	====

Figure 7.9.12-1

The total factored pile axial loading must be less than ϕR_n for the pile design. Designers should note that the driving resistance might be greater than the design loading for liquefied soil conditions. This is not an overdriving condition. This is due to the resistance liquefied soils being ignored for design, but included in the driving criteria to place the piles.

Method II (Technique I) - Matrix Coefficient Definitions

The stiffness matrix, shown in Figure 7-B-1.b, containing the spring values and using the standard coordinate system is shown in Figure 7-B-1.a. The sign of all the terms must be determined based on the sign convention.
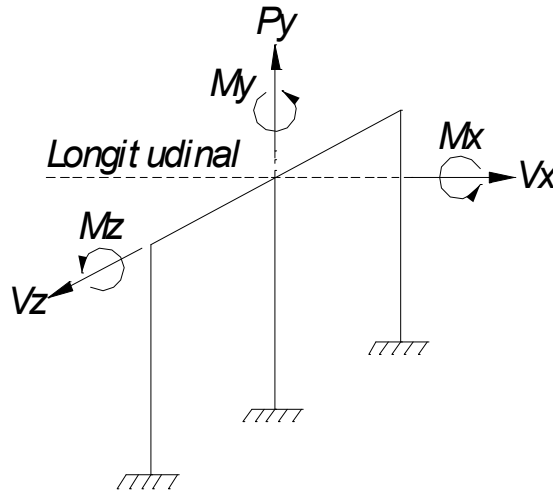
Standard Global Matrix
Figure 7-B-1.a

$$\begin{Bmatrix} & Vx & Py & Vz & Mx & My & Mz \\ Vx & K11 & 0 & 0 & 0 & 0 & K16 \\ Py & 0 & K22 & 0 & 0 & 0 & 0 \\ Vz & 0 & 0 & K33 & K34 & 0 & 0 \\ Mx & 0 & 0 & K43 & K44 & 0 & 0 \\ My & 0 & 0 & 0 & 0 & K55 & 0 \\ Mz & K61 & 0 & 0 & 0 & 0 & K66 \end{Bmatrix} \times \begin{Bmatrix} Disp. \\ \Delta x \\ \Delta y \\ \Delta z \\ \theta x \\ \theta y \\ \theta z \end{Bmatrix} = \begin{Bmatrix} Force \\ Vx \\ Py \\ Vz \\ Mx \\ My \\ Mz \end{Bmatrix}$$

Standard Global Matrix
Figure 7-B-1.b

Where the linear spring constants or K values are defined as follows using the Global Coordinates:

$K11 = +V_{x(app)}/+\Delta_x$ = Longitudinal Lateral Stiffness (kip/in)
$K22 = AE/L$ = Vertical or Axial Stiffness (k/in)
$K33 = -V_{z(app)}/-\Delta_z$ = Transverse Lateral Stiffness (k/in)
$K44 = +M_{x(app)}/+\theta_x$ = Transverse Bending or Moment Stiffness (kip-in/rad)
$K55 = JG/L$ = Torsional Stiffness (kip-in/rad)
$K66 = +M_{z(app)}/+\theta_z$ = Longitudinal Bending or Moment Stiffness (kip-in/rad)
$K34 = -Vz(ind)/+\theta_x$ = Transverse Lateral Cross-couple term (kip/rad)
$K16 = +V_{x(ind)}/+\theta_z$ = Longitudinal Lateral Cross-couple term (kip/rad)
$K43 = +M_{x(ind)}/-\Delta_z$ = Transverse Moment Cross-couple term (kip-in/in)
$K61 = +M_{z(ind)}/+\Delta_x$ = Longitudinal Moment Cross-couple term (kip-in/in)

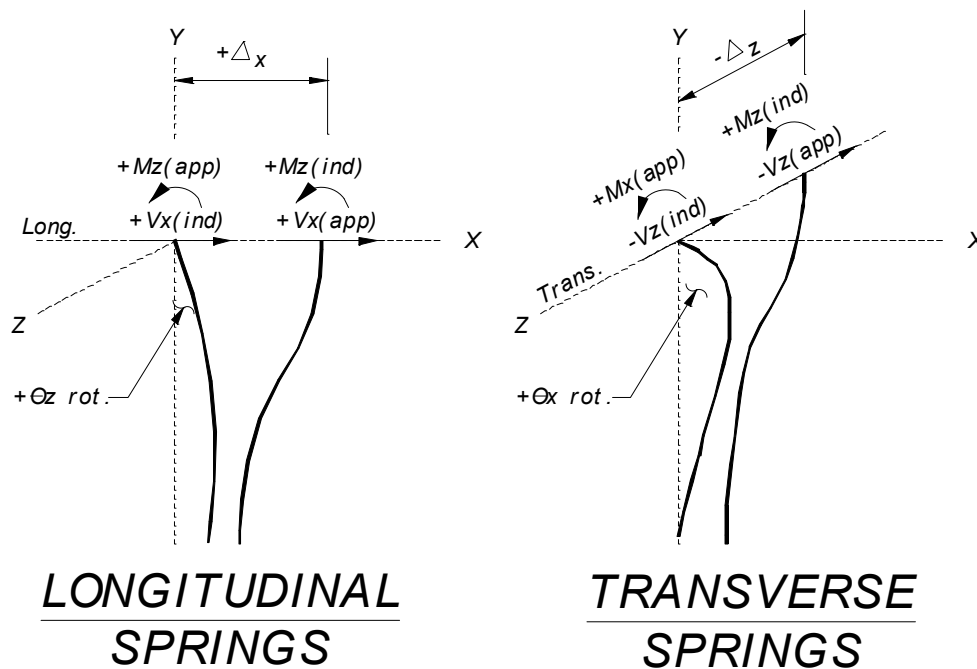## Fixed Head vs. Free Head Spring Calculations

Fixed Head

If the shear and moment are creating deflection in OPPOSING directions where the spring is located, a fixed head boundary condition is required to model the loaded foundation in a finite element model. See Figure 7-B-1.c for the fixed head coordinate system assumed in the following spring calculations.

Since applying load to a fixed end results in no reaction, a soil/structure interaction analysis will generally analyze the shear and moment simultaneously as a free head. Using the soil response results, a cross-couple correction term will be required in a FEM to produce the induced moment in the element modeling the fixed head condition. If accurate stresses in fixed head element are not required, the cross-couple term may be omitted.

There are two ways to model fixed head pile group. The most common method for a column footing is to use a group spring to model a group of piles or shafts as one set of springs. This method uses six linear springs to represent the foundation behavior. Lateral loads resisted by Cross-couples terms do not apply and individual pile loads must be calculated from the FEM results.

The second method would be to model the individual piles. This is more helpful for analyzing local stresses in the foundation cap element and for each pile. Cross-couple terms may be included and individual pile loads are generated in the FEM.

Fixed Head Coordinate System
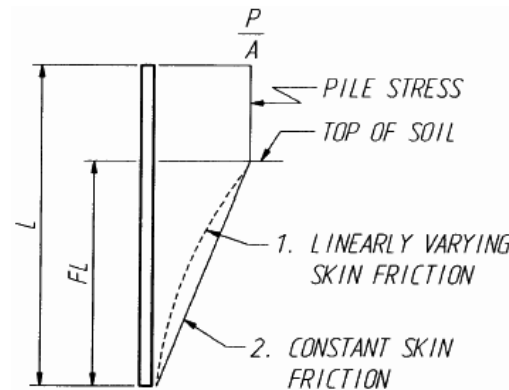*Figure 7-B-1.c*

Free Head

If the shear and moment are creating deflection in the SAME direction where the spring is located, a free head boundary condition is required to model the loaded foundation in a finite element model. If a free head boundary condition is assumed Method II (Technique II) described in BDM Section 7.2.5 must be used.

Vertical Springs (K22)

Vertical spring constants can be calculated from the following three assumptions. See Figure 7-B-1.e and the following definitions. REF: Page 6-30, Seismic Design of Highway Bridges Workshop Manual, Pub. No. FHWA-IP-81-2, Jan 1981.

A = Cross sectional area (in^2)
E = Young's modulus (ksi)
L = Length of pile (in)
F = Fraction of pile embedded

Pile Stress
Figure 7-B-1.d

Point Bearing Piles:

$$K22 = \frac{AE}{L}$$

Friction Piles w/ linearly varying skin friction:

$$K22 = \frac{AE}{\left(1 - \frac{2F}{3}\right)L}, \text{ with F = 1.0 (fully embedded), } K22 = 3\frac{AE}{L}$$

Friction Piles w/ constant skin friction:

$$K22 = \frac{AE}{\left(1 - \frac{F}{2}\right)L}, \text{ with F = 1.0, (fully embedded), } K22 = 2\frac{AE}{L}$$

Torsional Springs (K55)

The DFSAP program calculates acceptable torsional spring values for shafts and may be used for foundation springs. In general, torsional spring constants for individual piles are based on the strength of the pile. The statics equation for torsional resistance is given below.

$$K55 = \frac{M}{\phi} = \frac{T}{\phi} = \frac{JG}{L} \quad \text{where,}$$

$G = 0.4E$
$J = \text{Torsional Moment of Inertia}$
$L = \text{Length of Pile}$

Lateral Springs (K11 & K33)

A fixed head lateral spring can be found by applying the shear and axial load in a soil response program with the rotation at the top equal to zero and finding the lateral deflection that results. The spring value is the applied shear divided by the resulting deflection.

$$K11 = \frac{V_{x(app)}}{\Delta_x} \ (\text{longitudinal})$$

$$K33 = \frac{V_{z(app)}}{-\Delta_z} \ (\text{transverse})$$

Rotational Springs (K44 & K66)

Ideally a fixed head boundary condition would result in no rotation. Therefore K44 & K66 would be infinitely stiff.

In the past the fixed head rotational springs where found by applying the moment and axial load in a soil response program with the translation at the top equal to zero and finding the rotation that results. The spring value is the applied moment divided by the resulting rotation.

$$K44 = \frac{M_{x(app)}}{\theta_x} \ (\text{transverse}) \qquad K66 = \frac{M_{z(app)}}{\theta_z} \ (\text{longitudinal})$$

Cross-Couple Springs (K16, K34, K43 & K61)

Fixed Head

Cross-couple springs will not be symmetric for non-linear modeling foundation modeling. Since finite element programs will use matrix multiplication to generate reactions, doing the math is the easy way to show the effect of cross-couple terms. Note that K16 and K34 terms will have opposite signs.

$$
\begin{Bmatrix}
 & Vx & Py & Vz & Mx & My & Mz \\
Vx & K11 & 0 & 0 & 0 & 0 & K16 \\
Py & 0 & K22 & 0 & 0 & 0 & 0 \\
Vz & 0 & 0 & K33 & K34 & 0 & 0 \\
Mx & 0 & 0 & K43 & K44 & 0 & 0 \\
My & 0 & 0 & 0 & 0 & K55 & 0 \\
Mz & K61 & 0 & 0 & 0 & 0 & K66
\end{Bmatrix}
\times
\begin{Bmatrix}
Disp. \\
\Delta x \\
\Delta y \\
\Delta z \\
\theta x \\
\theta y \\
\theta z
\end{Bmatrix}
=
\begin{Bmatrix}
Force \\
Vx \\
Py \\
Vz \\
Mx \\
My \\
Mz
\end{Bmatrix}
$$

The longitudinal reactions are:

$$V_x = K11 \cdot \Delta_x + K16 \cdot \theta_z \text{ and } M_z = K61 \cdot \Delta_x + K66 \cdot \theta_z$$

The transverse reactions are:

$$V_z = K33 \cdot \Delta_z + K34 \cdot \theta_x \text{ and } M_x = K43 \cdot \Delta_z + K44 \cdot \theta_x$$

For a true fixed head boundary condition (translation only) in the X and Z directions, there will be no rotation about the X and Z axis. θ_x and θ_z will be zero (or approach zero). This means the K34 and K16 cross-couple terms will not affect the shear reactions. Likewise, the K66 and K44 rotational terms zero out and do not effect the moment reaction. This leaves the K61 and K43 cross-couple terms to generate induced moments based on the deflections in the X and Z directions. Designers should note, the cross-couple moments are applied to a fixed footing element and are resisted axially by the piles. This affects the local stress in the footing and axial loading of the pile much more than the column moment and shear, which is usually the primary focus for design.

K11 and K66 (or K33 and K44) alone do not predict the shape or reaction of the foundation element. The cross-couple term K16 (or K34) will add a shear force to correct the applied moment deflection.

Modeling real life features may be somewhat different than the theoretically true fixed condition. The top of a column at the superstructure or some pile and shaft applications may have opposing shear and moment, however the moment may be much less than the theoretical induced free head moment value. In other words, there may be significant rotations that need to be accounted for in the spring modeling. Designers need to be aware of this situation and use engineering judgment. The FEM would have rotations about the X and Z axis. θ_x and θ_z will NOT be zero and both the cross-couples terms and rotational springs may significantly affect the analysis.

The spring value for the lateral cross-couple term is the induced shear divided by the associated rotation.

$$K16 = \frac{V_{x(ind)}}{\theta_z} \text{ (longitudinal)} \qquad\qquad K34 = \frac{-V_{z(ind)}}{\theta_x} \text{ (transverse)}$$

The spring value for the moment cross-couple term is the induced shear divided by the associated rotation.

$$K61 = \frac{M_{x(ind)}}{-\Delta_z} \text{ (longitudinal)} \qquad\qquad K43 = \frac{M_{z(ind)}}{\Delta_x} \text{ (transverse)}$$

Method III – Non-Linear Springs

A finite element model may use non-linear springs based on PY curves to represent foundation response as shown in Figure 7-B-2.a. PY curves graph the relationship between the lateral soil resistance and the associated deflection of the soil. Generally, P stands for a force per unit length (of pile) such as kips per inch. Y is the corresponding horizontal deflection (of pile) in units such as inches.
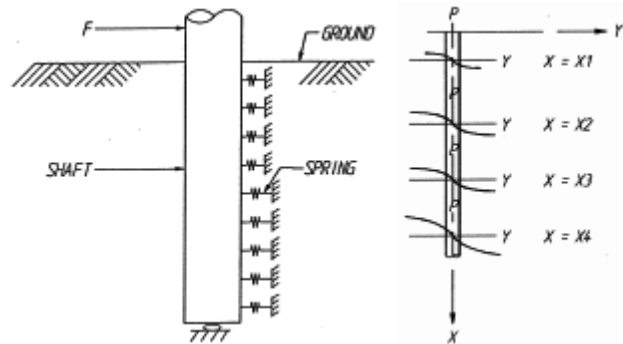
Pile Model using a Set of Non-linear PY Curves
Figure 7-B-2.a

Node placement for springs should attempt to imitate the soil layers. Generally, the upper ⅓ of the pile in stiff soils has the most significant contribution to the lateral soil reaction. Springs in this region should be spaced at most 3 feet apart. Spacing of 2.5 feet has demonstrated results within 10% of Lpile output moment and shear. Springs for the lower ⅔ of the pile can transition to a much larger spacing. Stiff foundations in weak soils will transfer loads much deeper in the soil and more springs would be sensible.

Transverse and longitudinal springs must include group reduction factors to analyze the structure/soil response. Soil properties are modified in Lpile to account for Group Effects. Lpile then generates PY curves based on the modified soil properties and desired depths. See BDM Section 7.2.5 for Group Effects.

FEM programs will accept non-linear springs in a Force (F) vs. Deflection (L) format. P values in a PY curve must be multiplied by the pile length associated with the spring in the FEM. This converts a P value in Force/Length units to Force. This cannot be done during dynamic analysis with some FEM programs (including GTStrudl).

Soil Modulus - E_S

Soil Modulus is defined as the force per length (of a pile) associated with a soil deflection. As shown in Figure 7-B-2.b, E_S is a slope on the PY curve or P/Y. E_S is a secant modulus since the PY relationship is nonlinear and the modulus is a constant. The units are F/L per L or F/L^2, such as kips per square inch.

Subgrade Modulus - k_S

A closely related term is the Subgrade Modulus (or Modulus of Subgrade Reaction) provided in a Geotechnical Report. This is defined as the soil pressure associated with a soil deflection. The units are F/L^2 per L or F/L^3, such as kips per cubic inch.
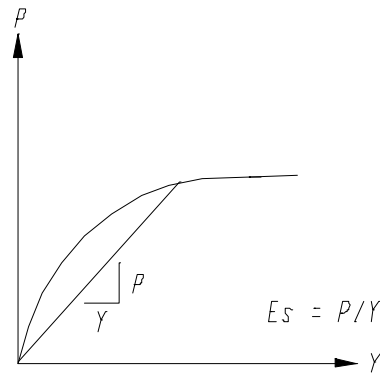
Secant Modulus Illustration
Figure 7-B-2.b

Method II (Technique I) – Pile Footing Matrix Example

A matrix with cross-couple terms is a valid method to model pile supported footings. The analysis assumes the piles will behave similar to a column fixed at the bottom (in the soil) with lateral translation only at the top (no rotation). This requires Fixed Head Boundary Condition to calculate spring values.

The Lpile program will solve for non-linear soil results for individual piles. See Group Effects in BDM Section 7.2.5 to reduce the soil properties of a pile in a group in both the transverse and longitudinal directions. This sample matrix calculates a foundation spring for an individual pile.

If a pile group has a large number of piles, the GPILE computer program is available to generate a spring matrix for the group. The program also computes individual pile loads and deflections from input loads. The output will contain a SEISAB {6 x 6} stiffness matrix. GTStrudl or SAP matrices have the same coefficients with a different axis orientation for the pile group.

The pile spring requires eight pile stiffness terms for a matrix as discussed in BDM Appendix 7-B-1. The following sample calculations discuss the lateral, longitudinal, and cross-couple spring coefficients for a GTStrudl local coordinate system. See Appendix 7-B-1 for axial and torsion springs.

The maximum FEM transverse and longitudinal seismic loads (Vy, Mz, Vz, My and axial Px) provide two loads cases for analysis in Lpile. The Lpile results of these two load cases will be used to calculate lateral, longitudinal, and cross-couple spring coefficients.

This sample calculation assumes there are no group effects. Only the longitudinal direction will be calculated, the transverse direction will be similar. A standard global coordinate system is assumed for the bridge. This sample will also assume a GTStrudl element is used to provide the foundation spring, which requires a different local axis coordinate system to input matrix terms, as shown in Figure 7-B-3.a. When the coordinate system changes, the sign convention of shear and moment also will change. This will be expressed in a 6x6 matrix by changing the location of the spring values and in sign of any cross-couple terms.
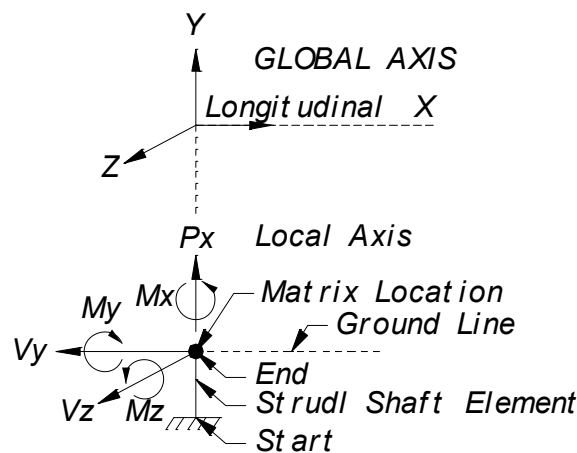
GTStrudl Local Axis Coordinate System
Figure 7-B-3.a

The locations of GTStrudl matrix terms are shown in Figure 7-B-1.b. The displacements are local and this requires the spring coefficients to be moved to produce the correct local reactions. The X axis is the new vertical direction. The Y axis is the new longitudinal direction. The spring coefficient definitions and notation remains the same as defined in BDM Appendix 7-B-1. Note the shift in diagonal terms and locations of the cross-couple terms.

$$
\begin{Bmatrix}
 & Px & Vy & Vz & Mx & My & Mz \\
Px & K22 & 0 & 0 & 0 & 0 & 0 \\
Vy & 0 & K11 & 0 & 0 & 0 & K16 \\
Vz & 0 & 0 & K33 & 0 & K34 & 0 \\
Mx & 0 & 0 & 0 & K55 & 0 & 0 \\
My & 0 & 0 & K43 & 0 & K44 & 0 \\
Mz & 0 & K61 & 0 & 0 & 0 & K66
\end{Bmatrix}
\times
\begin{Bmatrix}
\text{Disp.} \\
\Delta x \\
\Delta y \\
\Delta z \\
\theta x \\
\theta y \\
\theta z
\end{Bmatrix}
\equiv
\begin{Bmatrix}
\text{Force} \\
Px \\
Vy \\
Vz \\
Mx \\
My \\
Mz
\end{Bmatrix}
$$

GTStrudl Matrix in Local Coordinate System
Figure 7-B-3.b

Where the linear spring constants or K values are defined as follows (see Figure 7-B-3.c for direction and sign convention)

$K11 = -V_{y(app)}/-\Delta_y$ = Longitudinal Lateral Stiffness (kip/in)
$K22 = AE/L$ = Vertical or Axial Stiffness (k/in)
$K33 = -V_{z(app)}/-\Delta_z$ = Transverse Lateral Stiffness (k/in)
$K44 = -M_{y(app)}/-\theta_y$ = Transverse Bending or Moment Stiffness (kip-in/rad)
$K55 = JG/L$ = Torsional Stiffness (kip-in/rad)
$K66 = M_{z(app)}/\theta_z$ = Longitudinal Bending or Moment Stiffness (kip-in/rad)
$K34 = -V_{z(ind)}/-\theta_y$ = Transverse Lateral Cross-couple term (kip/rad)
$K16 = -V_{y(ind)}/+\theta_z$ = Longitudinal Lateral Cross-couple term (kip/rad)
$K43 = -M_{y(ind)}/-\Delta_z$ = Longitudinal Moment Cross-couple term (kip-in/in)
$K61 = +M_{z(ind)}/-\Delta_y$ = Transverse Moment Cross-couple term (kip-in/in)
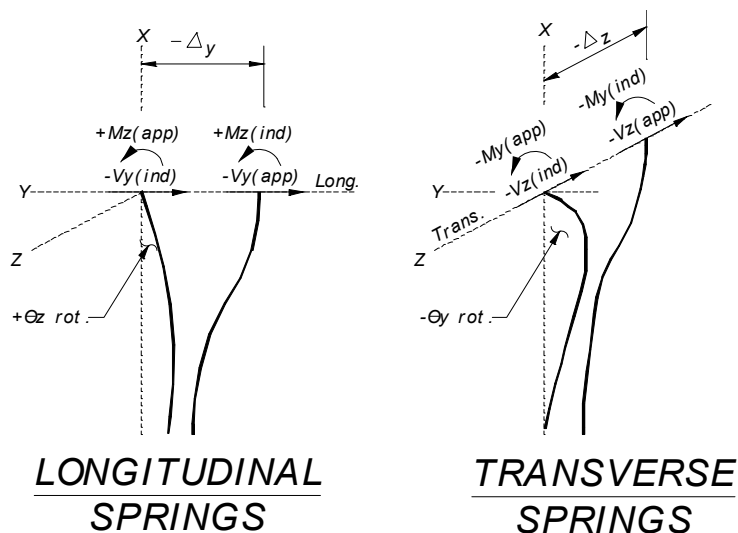
GTStrudl Local Coordinate System
Figure 7-B-3.c

Results from GTStrudl (local coordinate system)

P_x = 50,000 lbs   (axial load)
V_y = -60,000 lbs   (shear along longitudinal axis)
V_z = -40,000 lbs   (shear along transverse axis)
M_y = -2,230,000 lb-in   (moment about longitudinal axis)
M_z = 3,350,000 lb-in   (moment about transverse axis)

Load Case 1 - Longitudinal Direction

Load case 1 applies the lateral load (V_y) and axial load (P_x), and restrains the top against rotation (slope = 0 rad).

Input to Lpile:

Boundary condition code   =   2
Lateral load at the pile head =   -60000.000 lbs
Slope at the pile head   =   0.000 in/in
Axial load at the pile head   =   50000.000 lbs

Output from Lpile:

X	Deflection Δ_y	Moment $M_{z(ind)}$	Shear $V_{y(app)}$	Slope
in	in	lbs-in	lbs	Rad.
*****	**********	**********	**********	**********
0.000	-0.13576	3.761E+06	-60000.000	0.000000

Load Case 2 - Longitudinal Direction

Load case 2 applies the moment load (M_z) and axial load (P_x), and restrains the top against deflection (deflection = 0 rad).

Input to Lpile:

Boundary condition code   =   4
Deflection at the pile head   =   0.000 in
Moment at the pile head   =   3.350E+06 in-lbs
Axial load at the pile head   =   50000.000 lbs

Output from Lpile:

X	Deflection	Moment $M_{z(ind)}$	Shear $V_{y(ind)}$	Slope θ_z
in	in	lbs-in	lbs	Rad.
*****	**********	**********	**********	**********
0.000	0.00000	3.350E+06	-33027.667	0.001192

Springs Constants – Longitudinal Direction

$K11 = -V_{y(app)}/-\Delta_y$ $\quad\quad$ = -60 kip/-0.13576 in $\quad\quad$ = 442 kip/in
$K66 = M_{z(app)}/\theta_z$ $\quad\quad$ = 3,350 kip-in/0.001192 rad $\quad$ = 2,810,403 kip-in/rad
$K16 = -V_{y(ind)}/+\theta_z$ $\quad\quad$ = -33 kip/0.001192 rad $\quad\quad$ = -27,685 kip/rad
$K61 = +M_{z(ind)}/-\Delta_y$ $\quad\quad$ = 3,761 kip-in/-0.13576 in $\quad$ = -27,703 kip-in/in

$$
\left\{\begin{array}{ccccccc}
 & Px & Vy & Vz & Mx & My & Mz \\
Px & K22 & 0 & 0 & 0 & 0 & 0 \\
Vy & 0 & 442\dfrac{kip}{in} & 0 & 0 & 0 & -27,685\dfrac{kip}{rad} \\
Vz & 0 & 0 & K33 & 0 & K34 & 0 \\
Mx & 0 & 0 & 0 & K55 & 0 & 0 \\
My & 0 & 0 & K43 & 0 & K44 & 0 \\
Mz & 0 & -27,703\dfrac{kip-in}{in} & 0 & 0 & 0 & 2,810,403\dfrac{kip-in}{rad}
\end{array}\right\}
\times
\left\{\begin{array}{c}
Disp. \\ \Delta x \\ \Delta y \\ \Delta z \\ \theta x \\ \theta y \\ \theta z
\end{array}\right\}
\equiv
\left\{\begin{array}{c}
Force \\ Px \\ Vy \\ Vz \\ Mx \\ My \\ Mz
\end{array}\right\}
$$